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# Slate Waste
# aggregate for unbound pavement layers

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## ABSTRACT

The use of slate waste as an alternative to conventional aggregates in the construction of roads helps to conserve the supplies of premium grade aggregates and assists in problems arising from disposal of the waste.

Building on the foundation of previous experience with slate waste the properties and applications of this material as an unbound granular layer in road works have been studied. The study has involved laboratory testing of slate aggregate to determine mechanical and physical properties. The effects of varying particle gradation have been explored with respect to permeability, moisture-density relationship and frost susceptibility.

The bearing capacity, compaction and degradation of slate granular material in situ has also been studied in some detail.

Slate waste sources in North Wales, Mid Wales and the Lake District have been studied and the variation in their properties documented.

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# 1 INTRODUCTION

The construction industry has become aware that it is consuming increasing quantities of raw materials at the same time as seeking to conserve its natural resources (1). There is increasing recognition of the fact that this paradox can be partly resolved by more efficient recycling of industrial by-products and the use of waste materials (2).

It is now becoming recognized that there are localized shortages of conventional road building materials, particularly in urban areas. These shortages are critical in some cases and it is anticipated that the problem will both intensify and become more widespread (3).

The construction and reconstruction of highways consumes large quantities of unbound granular materials and it is perhaps in this area that slate waste could be used most effectively. Serious attention has therefore been paid to the use of slate waste as an unbound granular material in road construction in this dissertation.

There have been a few publications covering this field (4,5,6) and these have indicated that there is some potential, but the actual usage of slate waste has been generally rather limited apart from some instances where advantage has been taken of local circumstances (7,8).

There is a notable lack of reference material or even broadly based review work giving a comprehensive coverage of the subject. Consequently those who are in a position to use waste slate but are not materials specialists often find difficulty in relating slate waste to specification requirements or pavement performance.

The objectives of this dissertation are outlined below:

- to review the geological history of slate and its influence on the engineering properties and current utilisation of slate waste.

- to identify relevant standards and other specifications relevant to aggregate for unbound pavement layers.

- to determine the physical and mechanical properties of slate aggregate and the performance of these aggregates in granular materials.

- to assess the in-situ performance of slate waste used as an unbound pavement layer.

- to identify the main factors influencing slate aggregate behaviour in various pavement constructions and environmental conditions.

- to discuss the compliance of slate waste with current specifications and to explore some of the problems commonly encountered in applying these specifications to slate materials.

In order to fulfil these objectives slate aggregate and slate granular materials were tested in the laboratory. The in-situ performance of slate waste used in an unbound pavement layer was assessed during the reconstruction of the Bangor By-Pass.

Amongst other findings in this dissertation it is concluded that, with careful selecting and crushing, slate waste produces a material which performs well and complies with the requirements of current specifications for unbound pavement layers. If environmental issues concerning the use of waste materials are taken into consideration, slate waste appears to have a bright future as an aggregate for unbound pavement layers.

# 2 BACKGROUND

## 2.1 BRIEF HISTORY OF THE SLATE INDUSTRY

The earliest reference to the use of slate for roofing is the construction of a chapel roof at Bradford-on-Avon, England in the eighth century (9). Such slates were thick and rough, the workers not having attained the skill in splitting and trimming which characterised later practice.After the latter part of the 18th Century the industry attained considerable importance, particularly in areas of North Wales, Cornwall and the Lake District (10). During those early years the methods of abstraction and production were crude and wasteful. After about 1850, the growth of the Welsh slate industry was rapid. Large villages grew up around the quarries, and seaports were developed for transportation of the product. The slate industry has declined from a peak at the end of the 19th Century and is now very small. As a total waste of 75% to 90% of gross production was common there remains a considerable amount of waste in old tips.

## 2.2 THE GEOLOGY OF SLATE

Slate results from the alteration of materials such as clay, mudstone or shale, which usually originated as mud deposited on the floor of a sea or in a lake.

The nature of the alteration is such that the constituent particles are more closely packed in slate than they were in the original sediments. In slate the particles all lie in the same direction and tend to overlap in the manner of the scales on a fish.

The result of the alteration is that while clay quickly breaks down into mud when mixed with water (shale and mudstone less readily) slate is unaffected by it. Dry clay crumbles into fragments of irregular shape and size; shale can be split along planes determined by the original stratification of the rock, yielding weak brittle laminae, but slate can be split (in a direction that is not related to the bedding of the rock from which it was formed) into strong sheets of almost any desired thickness. This property is known as cleavage. The cause of the alteration is to be found in the movements which have from time to time taken place in different parts of the earth's crust, as a result of which rocks have been compressed and mountain ranges have been raised. The present accessibility of slate is due to the destructive action of rain, wind, and frost on rocks exposed at the surface of the earth, and the removal of the resulting debris by gravity, glaciers, and rivers, leading in time to the exposure of rocks that had once been deeply buried.It follows from its mode of origin, that slate may be expected to occur in regions where the rocks show signs of much folding and compression.

Slate is most frequently found amongst the rocks of the older geological formations, for the older the rock the more likely it is to have been affected by earth movements and to have been preserved until exposed as a result of the wearing away of the strata that once covered it.

The kind of cleavage apparent in slate is known as slaty cleavage in order to distinguish it from the cleavage displayed by many crystalline minerals; the latter results from a regular grouping of the constituent molecules as the mineral crystallises out from solution or from a molten condition. The cleavage of minerals may have an independent effect on the fracture of any of the constituent fragments or crystals in a rock, but slaty cleavage affects the rock mass uniformly as a whole.

Slaty cleavage has no relation to the bedding of the rock, that is to the planes separating the successive layers of sediment as originally deposited. In this respect slate differs from shale and certain kinds of sandstone and limestone as these rocks normally split along the bedding planes and the thickness of the sheets so produced is determined by that of the original layers.

## 2.3 THE PROBLEM OF SLATE WASTE

### 2.3.1 PROPORTION OF WASTE

A remarkable feature of the slate quarrying industry is the high percentage of waste material resulting from the production of roofing slates. In most quarries the waste averages 70 to 90 per cent of gross production (11). In view of this high percentage the problem of waste is of unusual importance.

### 2.3.2 CAUSES OF WASTE

Much of the waste is unavoidable and may be attributed to the following causes:

(i) The rock nearest the surface has usually suffered from long exposure to atmospheric pressure, ground water and frost which may have affected the rock along its joints, consequently not yielding blocks large enough to be of use.

(ii) Beds of poorly cleaved slate often have to be quarried in order to secure the good material with which they are associated.

(iii) Slate that is affected by closely spaced joints or has been shattered by blasting has to be discarded.

(iv) The cleavage of slate adjacent to dikes and sills of igneous rock has usually been impaired.

(v) Although the quarried blocks are reduced to a convenient size prior to conversion into roofing slates, the processes of splitting and trimming, especially the latter, produce large quantities of small fragments.

### 2.3.3. THE NEED FOR WASTE OUTLETS

As indicated by the enormous heaps of waste that are accumulated near slate quarries, the finished slate represents only a small proportion of the material actually quarried. Waste utilisation is therefore a matter of great importance to the slate industry.

Slate is forced to meet very keen competition from other types of roofing and structural materials and it is felt that any salvage from waste would better enable slate manufacturers to meet competition.

The need for an outlet for slate waste has been felt for many years. Various investigations have given more or less attention to the problem, but the results have been of little practical value (12, 13). Slate consists of various silicates that have few uses as compared with some other rock types. For example, limestone may be used for lime and cement manufacture, for agricultural purposes and furnace flux; all uses for which slate is unsuitable.

## 2.4 QUANTITIES AND LOCATION OF SLATE WASTE

From the foregoing it will be apparent the commercial adaptability is restricted and on this account all but a very small fraction of the waste accumulation since slate was first quarried is still lying in large waste heaps waiting for the discovery of fields for its utilisation.

The production of slate waste is now comparatively small but there is a legacy from the past of some 300 to 500 million tonnes. Of an annual waste production of a little over 1 million tonnes, only about 30,000 tonnes are used primarily as an inert filler powder in rubber, paints, paper, linoleum, etc., and as granules in roofing felt.

Table 2.1 gives recent estimates of current stockpiles of slate waste.

**Table 2.1 Quantities and location of slate waste.**

| LOCATION | EXISTING TIPS (x 1 million tonnes) |
|---|---|
| **WALES** | |
| Aberlleffni<br>Croes-y-Ddu-yr-Afon<br>Ffestiniog<br>Gilfach<br>Llechwedd<br>Maenoffern<br>Penrhyn<br>Pen-y-Orsedd | 300-500 |
| **LAKE DISTRICT** | |
| Burlington<br>Broughton Moor<br>Mandalls | 15-20 |
| **DEVON AND CORNWALL** | |
| Mill Hill<br>Old Delabole<br>Penpethy | 10-15 |

## 2.5 CURRENT UTILISATION OF SLATE WASTE

Waste from the primary production process has formerly been regarded as an unwanted commodity but attempts to discover uses for it have been made from time to time and have met with some measure of success (2).

Towards the end of the 19th Century a works was erected near one of the large quarries in North Wales to make bricks and tiles from slate waste, but although the products were dense and non-absorbent the industry did not thrive (11).

More recently, both in North Wales and Cornwall, efforts have been made to utilise the waste material by converting it into fine powder. The coarser powders are used as a "filler" for asphalt and the fine powders are used as a substitute for Fullers' earth and also as filler in uPVC pipeware (14, 15).

Slate powder is used as a filler in paint making. It is especially suitable in paint to be used as an undercoat for metallic surfaces, and it is used for Fullers' earth in degreasing (16).

Slate granules from 2-3 mm in size are used as a surfacing for roofing felt (14). Recent investment in this market has yielded coloured granules which has widened the market appeal. Crushed slate is also used with suitable resins to produce artificial roofing slates.

In recent years slate waste has been used as embankment fill, capping layers, pipe bedding, drainage materials and granular sub-base. Careful selection and crushing of the waste produces a material which complies with the requirements of Granular Sub-Base Type 1 and Type 2, in accordance with the "Specification for Highway Works" (17).

## 2.6 PREVIOUS EXPERIENCE WITH SLATE WASTE AS AN UNBOUND AGGREGATE

Sherwood, Tubey and Rose considered waste materials which occur in large quantities and which have high potential outlets in road construction in a report published in 1977 (6). They included slate waste among these materials and attributed the general lack of utilization of slate largely to difficulties in making it available at a given time in sufficient quantities at a reasonable price, but they noted that other factors may also be involved.

As far as slate waste is concerned they stressed the remote locations of the very large available quantity as the limitation to its value as a road making material; were it not for this factor they suggested that more research into its properties in this application would be justified. However they noted that it has been used for road works in areas where it is produced, and their report shows an embankment made from slate waste in a section of road improvement on Anglesey in North Wales. The flaky nature of the slate waste particles caused problems in compaction, and grid rollers were found to be useful since they break the elongated pieces of slate into shorter pieces; using this technique, slate waste has been successfully employed for sub base construction in road works in North Wales (7).

In this particular case very variable ground and ground water conditions imposed considerable difficulties in designing the road structure for the improvement of a two-lane road over a length of approximately 3 km. The design called for an overall road construction thickness of 800 mm comprising 520 mm of sub base, 190 mm road base and 90 mm of blacktop; this construction was required to give an even surface in the very variable conditions encountered.

The slate waste available had no cohesive fine fraction but did contain a proportion of large pieces. Crushing and/or screening were investigated as a means of eliminating material larger than 75 mm, but it was found that this was an uncertain as well as an uneconomic process; the slate pieces could be 500 mm long though still less than 75 mm across. The most suitable material was found to be waste machine dressing, comprising material generally below 100 mm, from which the occasional large pieces could be hand picked. Keeping the material at a low level in the road enabled the specification to be varied to accept a proportion of oversize pieces. It was agreed to use the material in the lower 350 mm of sub base where the question of susceptibility to frost damage would not arise.

The material was placed in two layers, each rolled with twelve passes of a grid roller and six passes of a vibrating roller; roll pressures were 0.73 Kg/mm and 2.33 Kg/mm respectively. The initial standard of rolling was determined by field sand replacement density tests and, from time to time in the course of the work, sand replacement tests were made as a check on the work.

As noted above, it was found that the action of the grid roller broke the longer needle-shaped pieces of slate into short pieces, thus improving compactability; furthermore, the process of breaking did not cause the material to become cohesive. Generally the fines in the material seemed to be in the sandy rather in the clayey range; and little trouble was experienced with compaction, even in very wet weather.

## 2.7 THE INTERNATIONAL USE OF SLATE WASTE IN ROAD CONSTRUCTION

In 1977 an OECD (Organisation for Economic Co-Operation and Development) report (18) noted that research and development had indicated the technical feasibility of using slate waste as an aggregate in sub base constructions as well as for embankment fill and improved sub-grades. The report reviewed the use of waste materials and by-products for road construction by twelve OECD members and indicated the broad usage of slate waste in these countries; unfortunately quantitative data was not available at the time. It is in fact understood that small quantities of slate waste are used for road foundation work in some industrial regions of France (19).

Although the overall relative consumption of slate waste for road construction may be small in the United States, one slate-producing company has found a good market for this application and the material is used by the Department of Highways and Transportation in Virginia (5). In Finland (20) slate quarries are located in the remote areas where little road construction activity exists and more suitable materials can usually be obtained readily. Furthermore, the extra cost of crushing the slate waste makes it even less competitive. Nevertheless, it has been used in some road construction applications in Finland, but only in local circumstances and for secondary roads. Although not included in the OECD report, Norway has a very considerable slate waste problem and the road construction industry is working hard to find applications for the material (20).

A report by Miller and Collins (1) was published in 1976 by the Transportation Research Board in the United States; this presented the results of a comprehensive survey of the technical and economic potential for making use of waste materials as alternative aggregates in highway applications. Of the initial group of 34 waste materials that were investigated, 31 were considered to show technical promise; and slate waste was among those that were regarded as deserving consideration for further development as aggregates.

According to this report slate waste is currently acceptable as a highway material in the United States and its principal use is as a granular base in highway construction. The report did not reveal any particular environmental factors which would tend to preclude or encourage the processing or actual use of slate waste in road construction; nevertheless, it was listed as one of the materials recommended for development as aggregate on the basis of environmental considerations.

The report referred to the strength, stability and durability of crushed slate and the undesirable flaky, elongated particle shape. It suggested that the actual use of slate waste in the United States had been very small and briefly discussed the usage in Virginia, noting that in 1972 a total of about 27,000 tonnes had been used there by the Department of Highways and Transportation.

In Virginia, slate waste is currently being used as an unbound granular material. According to one slate producer in Virginia (21), although the particle shape of the crushed slate waste is flaky and elongated, this has presented no more than token problems and that particular material has found a good market as a construction aggregate. Its relative hardness, compared with other slates in the United States, is important and many softer slates would probably fail to meet the necessary specifications.

## 2.8 FACTORS WHICH INHIBIT SLATE WASTE UTILISATION

It would seem to be a fairly straightforward exercise to evaluate slate as a resource in relation to its various potential applications. The main problem seems to be in the complex background of environmental, social, geographical and general economic factors involved (22).

A specific problem that is often quoted is the distance of many slate waste sources from potential markets. The sale of slate as an unbound granular material can not be undertaken successfully unless the transportation costs allow a competitive price. Obviously a specific proposal may depend on convenient access to a railway line or water, but due account must also be taken of factors such as unloading and offloading costs as well as the necessity or otherwise of having to make the journey in several stages by different means of transport.

The maximum economic transport distance depends on the nature of the product and the processing costs which have gone in to it. A report (1) on the use of the waste materials as potential replacements for highway aggregates in the United States has suggested that for this application, the limits are about 40 miles for road transportation and 100 miles and 300 miles for rail and water respectively. In the case of crushed slate for use as unbound granular material, this may be considerably less than 40 miles, depending on local circumstances. It may be argued that this suggests that there is little prospect for making widespread use of tips in this way. However, it should not be forgotten that the total area encompassed, for the sake of argument, by 30 mile radii drawn around every substantial tip in existence must be quite considerable; and the availability of portable processing plant may affect the extent to which small local tips can be used.

Apart from transportation, there are other considerations. It has been suggested that there is a stigma attached to the word "waste," (1) furthermore engineers are understandably happier working with established materials of known performance characteristics than with comparatively unknown materials which may give rise to unanticipated problems. It has also been said that specifications and codes of practice tend to be restrictive when the use of waste materials is envisaged.

## 2.9 THE ENVIRONMENT

An increasing awareness of the need to protect our environment from the despoliation and pollution caused by the disposal of waste materials and an increasing demand for raw materials, in particular for aggregates by the building and construction industry has focused attention on the contribution that waste materials can make to supplies. Utilisation of the old slate tips could contribute to the local economy and progressively restore the landscape.

It is our duty to look after our world prudently and conscientiously and to hand it on in good order for future generations. This is the often mentioned concept of "sustainable development". The exploitation of minerals provides a good opportunity to consider how this philosophy works in practice. Minerals are basic commodities widely used to create economic growth, but they are a finite resource. They are needed by society, but it is that same society which often opposes their extraction on the grounds of damage caused to the environment.

One of the reasons which influences this opposition is the legacy of old workings that present day society is frequently faced with. In other words the principles of sustainable development which were not adhered to in the past have now caught up with present day operators.

In a Transport and Road Research Laboratory Report published in 1974, Sherwood (23) pointed out that it is in the national interest to make use of lower grade and waste materials as alternatives to naturally occurring aggregates as this conserves the supplies of conventional aggregates and assists in problems arising from the disposal of unwanted materials. In addition, economic considerations make it likely that the cost of conventional aggregates will increase in those areas where they are already in short supply.

Sherwood showed that, in the United Kingdom, the total quantity of waste materials available far exceeds the amounts of aggregates and fill material required for roadworks, assuming that all wastes are suitable to act as substitutes for all the aggregates used in the pavement layers and for the materials used as imported fill. He pointed out that, of course, the scope for substitution is severely limited by the unsuitability of the wastes or by the non-availability within an economic haulage distance. Although the use of waste materials in road making can play a part in the accumulation of waste tips, he suggested that this is a relatively minor role; the major reasons for encouraging utilization are the conservation of conventional aggregates that can be put to better use elsewhere at a future date and the avoidance of disturbing the landscape (and possible consequent dereliction) by the excavation of further pits to obtain road making materials.

Environmental issues are now a matter of major local and national concern, specifically those relating to the depletion of non-renewable sources. Society has created a need for aggregates to construct its twentieth century roads but it also has a growing desire to maintain a natural environment. Due to the relative abundance of high quality granular materials in the United Kingdom there has not been the same incentive to use waste materials as there has been in other industrial countries.

For environmental considerations it is desirable that more waste should be used instead of natural resources. Such use produces benefit by reducing industrial dereliction, making land at present covered with spoil available for use, conserving supplies of natural resources and reducing the need to open more quarries and pits. Each of these benefits on their own would justify the use of waste and recycled granular materials and taken together they present an overwhelming case for encouraging wherever possible the use of such materials in the place of natural resources.

# 3 FUNCTIONS OF UNBOUND AGGREGATES IN ROADS

In the United Kingdom, the main uses of unbound aggregates are beneath the blacktop or concrete wearing/base courses, forming the sub-base, and if required, capping. Capping is essentially an improvement of the formation and is usually considered part of the earthworks.

The functions of unbound aggregates can be summarised as follows:

(i) to provide a working platform for construction traffic.

(ii) to provide a drainage layer

(iii) to contribute to the structural performance of the finished pavement.

(iv) to act as a frost blanket.

The provision of a working platform is an obvious requirement. A conflict is perceived between the drainage requirement and the structural requirement, the former is satisfied best by open gradings and the latter by dense. The structural contribution of the unbound layer to the performance of the overall finished pavement is usually modest. The need for a frost blanket depends upon the climate but has not been a major factor in the United Kingdom where the policy is not to allow frost susceptible material within the depth of frost penetration (currently specified as 450 mm from the surface) (17).

Another major use of unbound aggregates in highway engineering is in drainage layers and trenches where the principles are similar to those involved when the drainage function of the sub-base is considered. Geosynthetics are often used in conjunction with unbound aggregates to enhance performance.

To fulfil the above objectives the material must be properly compacted in a layer of suitable thickness to give appropriate drainage characteristics and adequate stiffness and strength. The particles which need to be sufficiently hard and durable, may be composed of natural materials, such as gravel or crushed rock, or of artificial materials such as industrial by-products, or processed wastes. In certain countries of mainland Europe, there is considerable pressure from legislation to make maximum use of recycled materials.

The types of materials which can be used, and their properties (for example, grading or particle strength) are prescribed in a specification. The mode of usage may also be specified either in terms of a stipulated method or by requiring a certain performance to be achieved. Compliance with material specifications involves laboratory testing, whilst the specification of performance (end result) limits, and the development of an end product specification requires in-situ testing.

# 4 CURRENT SPECIFICATIONS FOR UNBOUND AGGREGATES

## 4.1 GENERAL

There has never been a British Standard for granular materials. In 1986 the Department of Transport issued the "Specification for Highway Works" (SHW) (17) which is effectively the sixth edition of the specification for Roads and Bridge Works. The SHW was supplemented with "Notes for Guidance on the specification for Highway Works" (24). These documents are of primary importance as documents forming the basis for design and construction of motorways and trunk roads in the U.K. and are used also by local authorities. A seventh edition of the SHW appeared at the end of the preparation of this dissertation. It contains detailed differences to the clauses of relevance to slate, but there has not been time to discuss these. In general the changes are small. Relevant sections of the specifications (17) are given in Appendix G.

Specification for unbound aggregates is of a prescriptive nature requiring, or excluding, certain aggregate types, an envelope of particle size distribution (PSD), some specific aspects of the aggregate particles, e.g. hardness, and, as required, some general qualifying statements such as "well graded" etc.

Throughout the U.K. there is a wide variation in geological strata from which unbound aggregates can be extracted. Climatic conditions also vary and perhaps therefore it is surprising that there is such little flexibility in the case of sub base materials. Locally and regionally it is not uncommon for granular sub base Type 1 requirements to be modified to enable good use to be made of experience in the selection of aggregates able to fulfil their purpose whilst retaining desired engineering properties. This may be so in areas of the country where special local materials exist.

The concept of capping is to enable aggregates to be used from local sources or indigenous to the site and thus the specification is less exacting than for granular sub base Type 1 with any reduction in quality compensated by enhanced thickness.

The origin, and subsequent modifications, of many of the unbound aggregate particle size distribution envelopes and specification requirements is unclear. Although there are isolated instances of the use of end product performance or acceptance tests, the understanding of unbound aggregate behaviour is unlikely to be advanced until appropriate techniques are introduced to facilitate measurement of engineering properties.

## 4.2 COMPACTION REQUIREMENTS

The compaction of unbound granular materials must normally comply with a method specification such as that detailed in the SHW (17). This requires compliance with Clause 802 which includes Table 8/1 reproduced here as Table 4.1 which gives the compaction requirements for Type 1 and Type 2 granular sub base and also for wet mix macadam.

The compaction plant detailed in Table 4.1 is categorised in terms of static mass. The mass per metre width of roll is the total mass on the roll divided by the total roll width.

The surface of any layer of material shall, on completion of compaction of that layer, and before overlaying, be well closed, free from movement under the compacting plant and free from ridges, cracks, loose material, potholes, ruts or other defects. All defective areas shall be removed to the full thickness of the layer, and new material laid and compacted.

Many authorities and contractors prefer to carry out compaction to a performance specification using compacted density, air voids, plate bearing values, CBR values, or other tests to satisfy themselves that the material after compacting will behave satisfactorily. The SHW (17) allows for this by requiring that, if the method specified is not carried out, the contractor must satisfy and obtain approval from the Engineer, by demonstrating at site trials, that the state of compaction achieved by an alternative method is equivalent to or better than that using the specified method.

For an unbound material the difficulty of compaction is amongst other things a function of aggregate strength, shape of particles, grading and moisture content and therefore a performance specification is more desirable than a method specification. Also since it is difficult to recover undisturbed samples of compacted aggregates from site for laboratory testing, in situ procedures to evaluate compaction and mechanical properties are favoured.

# TABLE 4.1 Compaction requirements for granular material (17)

| Type of Compaction Plant | Category | Number of passes for layers not exceeding the following compacted thicknesses: | | |
|---|---|---|---|---|
| | | 110 mm | 150 mm | 250 mm |
| Smooth-wheeled roller (or vibratory roller operating without vibration) | Mass per metre width of roll: over 2700 Kg up to 5400 Kg | 16 | unsuitable | unsuitable |
| | over 5400 Kg | 8 | 16 | unsuitable |
| Pneumatic-tyred roller | Mass per wheel: over 4000 Kg up to 6000 Kg | 12 | unsuitable | unsuitable |
| | over 6000 Kg up to 8000 Kg | 12 | unsuitable | unsuitable |
| | over 8000 Kg up to 12000 Kg | 10 | 16 | unsuitable |
| | over 12000 Kg | 8 | 12 | unsuitable |
| Vibratory Roller | Mass per metre width of vibrating roll: Over 700 Kg up to 1300 Kg | 16 | unsuitable | unsuitable |
| | over 1300 Kg up to 1800 Kg | 6 | 16 | unsuitable |
| | over 1800 Kg up to 2300 Kg | 4 | 6 | 10 |
| | over 2300 Kg up to 2900 Kg | 3 | 5 | 9 |
| | over 2900 Kg up to 3600 Kg | 3 | 5 | 8 |
| | over 3600 Kg up to 4300 Kg | 2 | 4 | 7 |
| | over 4300 Kg up to 5000 Kg | 2 | 4 | 6 |
| | over 5000 Kg | 2 | 3 | 5 |
| Vibrating Plate Compactor | Mass per metre of base plate: over 1400g/sq metre up to 1800g/sq metre | 8 | unsuitable | unsuitable |
| | over 1800g/sq metre up to 2100g/sq metre | 5 | 8 | unsuitable |
| | over 2100g/sq metre | 3 | 6 | 10 |
| Vibro-tamper | Mass: over 50Kg up to 65Kg | 4 | 8 | unsuitable |
| | over 65Kg up to 75Kg | 3 | 6 | 10 |
| | over 75Kg | 2 | 4 | 8 |
| Power Rammer | Mass: 100kg up to 500Kg | 5 | 8 | unsuitable |
| | over 75Kg | 5 | 8 | 12 |

# 5 UNBOUND AGGREGATE LAYER DESIGN

The Design of roads in the U.K. generally follows Design Guides produced by the Transport and Road Research Laboratory (TRRL) (25), or the Department of Transport (DTp) (26,27). These design guides and any others produced by other procurement bodies or research organisations will for the purpose of describing unbound aggregates generally make reference to the SHW (17). In the period since its first publication the specification's requirements for unbound aggregates have 'evolved' to take account of previous observed performance - evolution based on experience of the prescribed types, therefore inappropriate possibly for materials which there is little or no experience.

The conventional basis of acceptance is firstly to meet all of the prescriptive requirements mentioned in Section 4, and secondly to place, compact and protect the aggregate as described in a 'method' style of specification (SHW) (17). If all of these factors have been achieved then the layer of unbound aggregate will be expected to fulfil its design role in terms of performance. Although direct performance measures such as modulus and permeability etc. could be adopted as a means of specification this 'end point' approach is not currently utilised widely in the U.K. A performance measure which is adopted is the requirement for the unbound aggregate not to be susceptible to the action of frost if placed within 450 mm of the road surface. The stiffness of a granular layer is strongly dependent upon the characteristic of the underlying material.

Analytical and performance monitoring techniques have been drawn together by Powell et.al. in TRRL Report LR 1132 'The Structural Design of Bituminous Roads' (1984) (25). Design curves are presented relating sub base thickness to CBR value of subgrade and construction traffic axles for a fixed limit of deformation at sub base level during construction.

Where soil conditions indicate a CBR value of less than 5% the sub base layer may be divided into:-

i) an upper layer of granular sub base Type 1 material overlying-

ii) an increased layer thickness of granular capping material.

At roadbase level and above aggregates bound with either bitumen or cement are generally employed but unbound aggregates could be used at this level in less heavily trafficked situations. Unbound aggregates for roadbase comprise Wet Mix Macadam (WMM) or Dry Bound Macadam (DBM). The DTp no longer permit unbound aggregates as roadbase materials in their roads and thus reference needs to be made to fifth, or earlier, editions of SHW (17) for specification details. WMM is included in LR 1132 (25) as a roadbase material but this and DBM are no longer in general use in the U.K. with the exception of minor local roads. Unbound bases or lightly treated materials are widely used in Continental Europe (e.g. France).

The need for a British Standard for Unbound Aggregates has been recognised and work had begun on its preparations. This has now been superseded by work on CEN (Comite European de Normalisation) (28) draft standards.

# 6 SLATE AGGREGATE TEST RESULTS

## 6.1 GENERAL

In order to assess the suitability of slate aggregates for unbound pavement layers a programme of testing was carried out on material from 6 quarries located as shown in Figure 6.1. The geological age of the quarries is detailed in Table 6.1.

TABLE 6.1 Geological age of aggregates from quarries used in the laboratory investigation.

| QUARRY | GEOLOGICAL AGE |
|---|---|
| Aberllefni | Ordovician |
| Burlington | Silurian |
| Croes-y-Ddu-yr-Afon | Ordovician |
| Ffestiniog | Ordovician |
| Llechwedd | Ordovician |
| Penrhyn | Cambrian |

The slate aggregates were tested in the laboratory for various mechanical, physical and chemical properties. Details are given in the remainder of this chapter.

In order to compare the test results of slate aggregates with conventional aggregates, a summary of means and ranges of test values for conventional aggregates is given in Table 6.2 (29).

## 6.2 DESCRIPTIVE TESTS

There are two quantitative measures of particle shape which may be included in the specification for aggregates for road construction. These are the flakiness index and elongation index.

Flakiness index tests were carried out in accordance with BS 812:Part 105.1:1989 (30), and elongation index tests in accordance with BS 812:Part 105.2:1990.(31). Test results are detailed in Appendix A. Flakiness index results ranged from 90-100%, and elongation index results from 20-38%. The aggregate tested from the Ffestiniog area indicated higher flakiness and higher elongation index values than the aggregates from the other quarries. Size fractions from 63.0 mm to 6.3 mm were tested for flakiness index and from 50.0 mm to 6.3 mm for elongation index. The overall view of the results did not reveal that any one particular size fraction was consistently different in its flakiness or elongation characteristics.

The surface texture of all the slate aggregate tested was determined to be smooth. This test is carried out by visual examination following guidelines detailed in BS 812:Part 1:1975 (32).

TABLE 6.2 Summary of means and ranges of test values for conventional aggregates (29)

| GROUP | MEASURE | Aggregate Crushing Value | Aggregate Impact Value | Water Absorption % | Relative Density |
|---|---|---|---|---|---|
| Basalt | Mean | 14 | 15 | 1.1 | 2.8 |
| | Range | 7 - 25 | 7 - 25 | 0.0 - 2.3 | 2.6 - 3.00 |
| Flint | Mean | 18 | 23 | 1.0 | 2.54 |
| | Range | 7 - 25 | 19 - 27 | 0.3 - 2.4 | 2.40 - 2.60 |
| Granite | Mean | 20 | 19 | 0.4 | 2.69 |
| | Range | 9 - 35 | 9 - 35 | 0.2 - 0.9 | 2.60 - 3.00 |
| Gritstone | Mean | 17 | 19 | 0.6 | 2.69 |
| | Range | 7 - 29 | 9 - 35 | 0.1 - 1.6 | 2.60 - 2.90 |
| Hornfels | Mean | 13 | 12 | 0.4 | 2.82 |
| | Range | 5 - 15 | 9 - 17 | 0.2 - 0.8 | 2.70 - 3.00 |
| Limestone | Mean | 24 | 23 | 1.0 | 2.66 |
| | Range | 11 - 37 | 17 - 33 | 0.2 - 2.9 | 2.50 - 2.80 |
| Porphyry | Mean | 14 | 14 | 0.6 | 2.73 |
| | Range | 9 - 29 | 9 - 23 | 0.4 - 1.1 | 2.60 - 2.90 |
| Quartzite | Mean | 16 | 21 | 0.7 | 2.62 |
| | Range | 9 - 25 | 11 - 33 | 0.3 - 1.3 | 2.60 - 2.70 |

## 6.3 WATER ABSORPTION

Water absorption tests were carried out in accordance with BS 812:Part 2:1975. (33) and are detailed in Table 6.3. All absorption values determined were very low, between 0.2 and 0.3% indicating that once placed the unbound aggregate would have a high durability.

## 6.4 PARTICLE STRENGTH

If strength is defined as the power possessed by an aggregate to resist fracture under an applied load, then the tests which are in common usage which are reflective of this quality are the aggregate crushing, ten per cent fines, and aggregate impact tests.

The aggregate crushing value is a measure of the resistance of an aggregate to crushing under a gradually increasing compressive load. The ten per cent fines value test is similar except that the force which causes ten per cent fines is determined. The aggregate impact value is carried out by subjecting an aggregate to 15 blows of a hammer. Each of these three tests are carried out on the 14-10 mm size fraction.

Table 6.3 Summary of slate aggregate test results.

| SOURCE | PENRHYN | FFESTINIOG | LLECHWEDD | CROES-Y-DDU-AFON | ABERLLEFENI | BURLINGTON |
|---|---|---|---|---|---|---|
| WATER ABSORPTION (%) | 0.2 | 0.3 | 0.3 | 0.3 | 0.2 | 0.3 |
| FLAKINESS INDEX(mean) | 93 | 100 | 100 | 100 | 93 | 98 |
| ELONGATION INDEX(mean) | 23 | 29 | 34 | 34 | 23 | 27 |
| AGGREGATE CRUSHING VALUE | 25 | 29 | 26 | 30 | 24 | 23 |
| AGGREGATE IMPACT VALUE | 27 | 29 | 29 | 33 | 28 | 28 |
| TEN PER CENT FINES VALUE (kN) | 160 | 130 | 140 | 120 | 170 | 160 |
| TEN PER CENT FINES VALUE (SOAKED) kN | 110 | 90 | 80 | 70 | 110 | 100 |
| OVEN DRY RELATIVE DENSITY | 2.80 | 2.76 | 2.77 | 2.75 | 2.80 | 2.80 |
| SATURATED SURFACE DRY RELATIVE DENSITY | 2.82 | 2.78 | 2.78 | 2.77 | 2.82 | 2.81 |
| APPARENT RELATIVE DENSITY | 2.84 | 2.79 | 2.80 | 2.79 | 2.84 | 2.83 |
| MAGNESIUM SULPHATE SOUNDNESS (%) | 99 | 98 | 98 | 98 | 99 | 98 |
| SLAKE DURABILITY INDEX (%) | 96 | 94 | 95 | 94 | 96 | 96 |
| PLASTICITY | NON PLASTIC | NON PLASTIC | NON PLASTIC | NON PLASTIC | NON PLASTIC | NON PLASTIC |
| SULPHATE CONTENT (g/Litre) | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 |

In all forms of flexible pavement the aggregate must be strong enough to support the weight of the rollers during construction and the repeated impact and crushing actions of traffic. Thus the aggregate must have a durable resistance to both crushing and impact. The strength tests just described are empirical attempts to measure this resistance; however, since they are empirical, the results obtained have to be correlated with field experience for them to have any significance.

An aggregate crushing test with a value greater than about 25 is generally taken to indicate a material which is too weak to be utilised in a pavement. Results from tests carried out on the slate aggregate from the different sources ranged from 23-30 (see Table 6.3) apparently indicating that the use of these aggregates may lead to weakness in the pavement.

When aggregates with aggregate crushing values greater than about 30 are evaluated, the crushing test begins to become insensitive due to the restrictive cushioning effect which the fines formed in the early part of the test have on later aggregate breakdown. The ten per cent fines test was developed to evaluate these weaker aggregates. The tests on the slate aggregates gave ten per cent values ranging from 120-170 kN when tested in the surface dry condition and from 70-110 kN when tested after being pre-soaked for 24 hrs. Values obtained with this test for conventional aggregates range from as low as 10 kN for chalk to over 400 kN for the hardest aggregates; normally, however, road aggregates should not show a fines value of less than 50 kN. Details are given in Table 6.3. As its name implies, the aggregate impact value provides a relative measure of the resistance of an aggregate to sudden shock, eg. as might occur under vibratory compaction. Values obtained with this test are, in general, numerically similar to those obtained with the aggregate crushing test. As it is a simple test to carry out, and uses comparatively unsophisticated equipment, the impact test therefore tends to be used instead of the more complicated crushing test. Values for aggregate impact tests carried out ranged from 27-33. ACV and AIV results on conventional materials are presented in Table 6.2. Test results are also detailed in Table 6.3.

Aggregate Crushing Value Tests were carried out in accordance with BS 812:Part 110:1990, (34). Aggregate Impact Value Tests in accordance with BS 812:Part 112:1990 (35), and Ten Per Cent Value Tests in accordance with BS 812:Part 111:1990 (36).

## 6.5 RELATIVE DENSITY

Relative density tests were carried out in accordance with BS 812:Part 2:1975 (33). Three values were determined on each aggregate; oven dried, saturated surface dried and apparent. The results which varied from 2.75 to 2.80 (oven dried), are higher than for most conventional aggregates (see Table 6.2). Results are detailed in Table 6.3.

## 6.6 SULPHATE SOUNDNESS

Magnesium Sulphate Soundness tests were carried out in accordance with BS 812:Part 121:1989 (37) on aggregate passing a 14.0 mm sieve and retained on a 10.0 mm sieve.

An aggregate is considered to be physically sound if it is adequately strong and is capable of resisting the influences of weathering, without disruption or decomposition. Mineral or rock particles that are physically weak, extremely absorptive, easily cleavable, or swell when saturated may be susceptible to breakdown through exposure to natural weathering processes. The use of such materials as unbound aggregate may reduce strength or lead to premature deterioration.

Shales, friable sandstones, some micaceous rocks, clayey rocks, some very coarse crystalline rocks and various cherts are examples of physically unsound aggregate materials; these may be inherently weak or may deteriorate through saturation, alternate wetting and drying, freezing, temperature changes, or by the disruptive forces developed as a result of crystal growth in the cleavage planes or pores.

The most important properties affecting physical soundness of aggregates are the size, abundance and continuity of pores and channelways within the particles. These pore characteristics influence freezing and thawing durability, strength, elasticity, abrasion resistance and specific gravity.

Because of the clay mineral origins of slate and the property of cleavage the soundness of slate aggregate is a concern amongst engineers. However as laboratory results show (see Table 6.3) the loss in weight after completion of the test is very low and is better than most conventional aggregates used as unbound material.

## 6.7 SLAKE DURABILITY

Slake durability tests were carried out in accordance with the procedures detailed in the International Society for Rock Mechanics publication "Rock Characterisation Testing and Monitoring" (38).

Comments at the beginning of Section 6.5 regarding the weathering processes of certain aggregates are also applicable to this section. The slake durability test is intended to assess the resistance offered by an aggregate to weakening and disintegration when subjected to two standard cycles of wetting and drying. The slake durability index values determined on slate aggregate in the laboratory indicate comparable resistance to disintegration by weathering to conventional high class unbound aggregates. Results are as detailed in Table 6.3.

## 6.8 PLASTICITY

The condition of a fine material can be altered by changing the moisture content: the softening of clay by the addition of water is a well-known example. For every clay soil there is a range of moisture contents within which the clay is of a plastic consistency and the Atterberg Limits (Liquid

Limit, Plastic Limit) provide a means of measuring and describing the plasticity range in numerical terms.

The SHW (17) has a requirement that the fine material (passing 425 micron) in Granular Sub-Base Material Type 1 should be non-plastic and for Granular Sub-Base Material Type 2 that the plasticity index of the fine material should be less than 6. The plastic limit can only be carried out on materials with some cohesion, therefore a material that is non-plastic indicates no cohesion and a low plasticity (i.e. $\leq 6$) indicates little cohesion. All the slate samples tested in accordance with BS 1377 (39) were determined to be non-plastic indicating suitability as Type 1 and Type 2 sub-base with regard to plasticity requirements.

## 6.9 SULPHATE CONTENT

The SHW (40) requires that material in contact with concrete has an aqueous sulphate content less than 2.5 g/litre. The slate samples when tested in accordance with BS 1377:1975:Test 10 (39) all indicated a sulphate content of 0.01 g/litre.

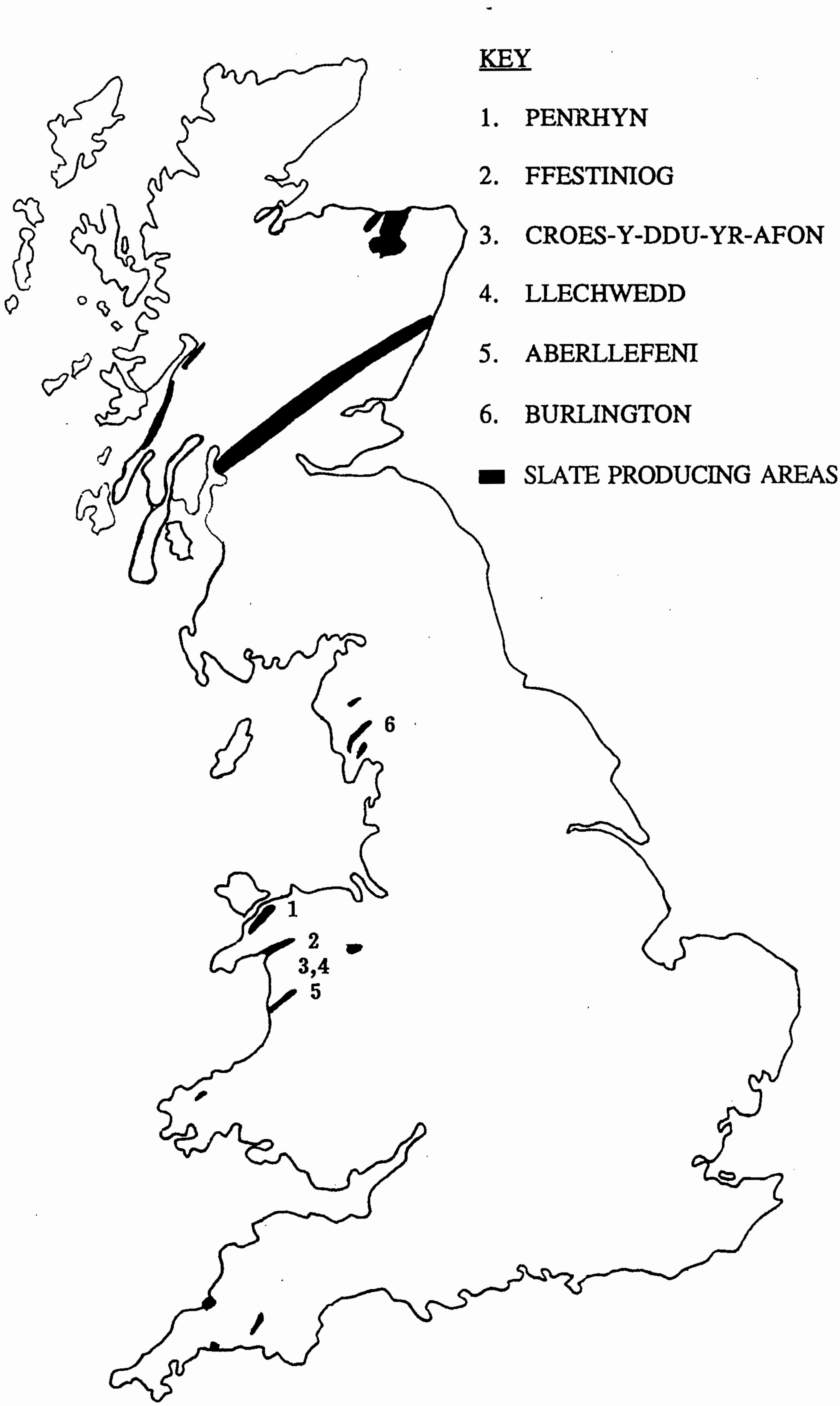



Figure 6.1 Locations of Slate Quarries

Figure 6.1 Locations of Slate Quarries

# 7 SLATE GRANULAR MATERIAL TEST RESULTS

## 7.1 GENERAL

On the basis of the test results described in the last chapter most slate aggregates can be seen to have similar properties. In order to determine their performance as granular materials one has been selected as a representative aggregate.

Samples of Granular Sub-Base Material Type 1 from Penhryn Quarry were prepared at different gradings in the laboratory to assess performance and to examine the influence of a change in grading on their properties.

The sample preparation involved screening bulk samples of sub-base into separate single size fractions, and then recombining these fractions to produce the required grading. This method allowed the grading produced to be strictly controlled and ensured that all tests were carried out on material having an identical grading. The series of grading curves produced covered the range over which the material could be produced at the quarry within SHW (17) Clause 803 specification Limits.

Details of the gradings produced in the laboratory are given in Table 7.1 and graphically presented on Figures 7.1 to 7.8.

The particle size distribution of an aggregate is usually expressed graphically in the form of a granulometric curve, the abscissae representing particle size to a logarithmic scale and the ordinate percentage by weight finer than the corresponding particle size.

The general slope, and shape of the curve is an indication of the grading or range of particle sizes of which the material is composed.

TABLE 7.1 Laboratory prepared gradings.

| SAMPLE REFERENCE | A | B | C | D | E | F | G | H | SHW SPECIFICATION LIMITS |
|---|---|---|---|---|---|---|---|---|---|
| BS SIEVE SIZE 75.0 mm | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 37.5 mm | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 85-100 |
| 20.0 mm | 64 | 67 | 70 | 73 | 76 | 79 | 84 | 87 | |
| 10.0 mm | 40 | 44 | 48 | 52 | 56 | 60 | 65 | 70 | 40-70 |
| 5.0 mm | 25 | 27 | 29 | 31 | 34 | 37 | 41 | 45 | 25-45 |
| 600 micron | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 8-22 |
| 75 micron | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 0-10 |

The granulometric curves do not give a complete picture of the effects that different samples from the same source can have on the results of other tests such as permeability and frost heave effects, but the curves do assist in understanding the results more clearly.

Gradation coefficients are generally used to classify the characteristics of a curve, but their use is empirical and thus of somewhat limited value. Definitions of the coefficients are given in Appendix B.

## 7.2 GRADING

Grading analysis, sieve analysis, screen analysis and mechanical analysis are synonymous terms which refer to the quantities, expressed as percentage by mass, of the various particle sizes of which a sample of aggregate is composed. These quantities are determined by separating the aggregates into portions which are retained on a number of sieves or screens having specified openings which are suitably graded from coarse to fine.

An indication of the control that can be achieved in the production of slate sub-base Type 1 is evident from an analysis of daily results of this material produced at Penrhyn Quarry during the last 4 years (41). For reasons which will be discussed later Penrhyn Quarry produce slate sub-base Type 1 to a narrower grading envelope than that allowed in the SHW (17), as indicated in Table 7.2.

A statistical analysis of daily results since the introduction of a quality assurance programme in 1988 has indicated that this narrower grading envelope has been achieved within 95% confidence limits (41).

Table 7.2 Penrhyn Quarry specification for Type 1 Sub-Base.

| BS SIEVE SIZE | PENRHYN QUARRY SPECIFICATION | SHW SPECIFICATION |
|---|---|---|
| 75.0 mm | 100 | 100 |
| 37.5 mm | 85-100 | 85-100 |
| 10.0 mm | 45-70 | 45-70 |
| 5.0 mm | 25-45 | 25-45 |
| 600 micron | 8-15 | 8-22 |
| 75 micron | 0-6 | 0-10 |

## 7.3 PERMEABILITY

The state of water movement in a granular material is named percolation, the measure of it called permeability and the factor relating permeability to unit conditions of control is called the coefficient of permeability.

The coefficient of permeability depends primarily on the average size and organisation of the pores, which in turn is loosely related to the distribution of particle sizes, particle shape and structure. In general, the smaller the size of the pores the lower is the coefficient of permeability. The presence of a small percentage of fines in an unbound granular material which makes the pores smaller results in a coefficient of permeability significantly lower than the value of the same material without fines.

Permeability tests were carried out in accordance with the constant head method detailed in BS 1377:1990:Part 4 (42), on material passing the 20 mm sieve at the eight different prepared gradings A-H. The results determined are as detailed in Table 7.3 and graphically presented in Figure 7.9.

TABLE 7.3 Constant head permeability test results at varying gradings

| SAMPLE REFERENCE | CONSTANT HEAD PERMEABILITY (M/sec) |
|---|---|
| A | $6.6 \times 10^{-2}$ |
| B | $2.4 \times 10^{-2}$ |
| C | $9.6 \times 10^{-3}$ |
| D | $4.7 \times 10^{-3}$ |
| E | $2.4 \times 10^{-3}$ |
| F | $8.2 \times 10^{-4}$ |
| G | $8.8 \times 10^{-4}$ |
| H | $6.7 \times 10^{-4}$ |

As can be seen from the results in Table 7.3 and Figure 7.9, permeability decreases with increasing fine material. It should be mentioned however that the testing of granular materials in this manner may give erroneous results. The problems associated with determining permeability in the laboratory are primarily caused because of the size of the mould relative to the aggregate size. The mould size used in the above series of tests was 150 mm diameter. As the maximum particle size used in the test was 20 mm this could have resulted in higher permeability test results being achieved than would be expected in a pavement layer, due to edge flow in the mould. This problem is discussed in greater detail in Section 9.2.

## 7.4 MOISTURE DENSITY RELATIONSHIP

As with permeability tests detailed in Section 7.3 the moisture-density relationship is affected by particle size distribution. Relationships were determined on the same 8 gradings in accordance with BS 1377: 1975: Test 14 (43) (vibrating hammer method).

Details of the optimum moisture contents and maximum dry densities are given in Table 7.4 and the relationships are presented graphically in Figures 7.10 to 7.17. The variation of maximum dry density and optimum moisture content with percentage passing 75 micron size sieve is plotted in Figure 7.18. The results indicate that peak dry density is achieved at about 5-6% passing 75 micron sieve, while optimum moisture content increases with increasing fines.

TABLE 7.4 Optimum moisture content and maximum dry density test results at varying gradings.

| Sample Reference | Optimum Moisture Content | Maximum Dry Density (kg/m$^3$) | Air Voids (%) | Pessimum Moisture Content (%) | Pessimum Dry Density (kg/m$^3$) |
|---|---|---|---|---|---|
| A | 4.8 | 2210 | 10.4 | 1.4 | 2190 |
| B | 4.8 | 2230 | 9.8 | 1.2 | 2200 |
| C | 4.9 | 2250 | 8.6 | 1.5 | 2210 |
| D | 5.1 | 2250 | 8.3 | 1.7 | 2210 |
| E | 5.3 | 2240 | 8.3 | 1.2 | 2220 |
| F | 5.5 | 2220 | 8.5 | 2.2 | 2200 |
| G | 5.5 | 2220 | 8.6 | 2.0 | 2200 |
| H | 5.6 | 2200 | 9.0 | 1.5 | 2190 |

## 7.5 FROST SUSCEPTIBILITY

The SHW (17) states that the method for determination of frost susceptibility given by TRRL Report LR 90 (44) and Supplementary Report SR 829 (45) shall be used to determine the frost susceptibility of granular materials. However both LR 90 and SR 829 have been subject to considerable criticism as the results produced are often conflictory and the methods have a poor repeatability.

In order to overcome these criticisms and to produce a more repeatable test method an alternative test was developed and this is detailed in BS 812: 1989: Part 124 (46). The BS 812 method has been experimentally proven to have a better repeatability and therefore is the preferred method. It seems likely that the BS 812 method will supersede the TRRL methods.

TABLE 7.5 Frost heave results on Type 1 Sub-base at varying densities

| SAMPLE REFERENCE | FROST HEAVE (mm) |
|---|---|
| A | 3 |
| B | 4 |
| C | 4 |
| D | 6 |
| E | 8 |
| F | 12 |
| G | 17 |
| H | 22 |

For the foregoing reasons the BS 812 procedure was chosen as the method of test for the determination of frost susceptibility.

The results obtained are detailed in Table 7.5 and the gradings of the samples after test are given in Appendix C. The mean change in grading of the slate sub base after testing is given in Table 7.6.

TABLE 7.6 Change in Gradings of Type 1 Sub-base material after frost heave test.

| BS Sieve Size | Mean of Original Gradings (%) Passing | Mean of Gradings after Frost Heave (%) Passing | Change in Grading |
|---|---|---|---|
| 75.0 mm | 100 | 100 | 0 |
| 37.5 mm | 100 | 100 | 0 |
| 20.0 mm | 74 | 77 | + 3 |
| 10.0 mm | 54 | 58 | + 4 |
| 5.0 mm | 34 | 39 | + 6 |
| 600 micron | 15 | 20 | + 5 |
| 75 micron | 6.7 | 10.1 | + 3.4 |

The graph of percentage material passing 75 micron versus frost heave plotted in Figure 7.19 indicates an obvious increase in heave with increasing fineness of the grading. Previous experience of slate indicated that this was likely to happen and it is typical of the behaviour of conventional aggregates.

## 7.6 CALIFORNIA BEARING RATIO

The California Bearing Ratio Test, or CBR test as it is usually called, is an empirical test, which was first developed in California, USA, for estimating the bearing value of highway sub-bases and subgrades. There are numerous ways of preparing samples for the test. The test samples prepared in the laboratory at the eight different gradings were compacted to maximum dry density at optimum moisture content as defined in Section 7.4 by use of a vibrating hammer.

Surcharge weights in the form of annular steel rings were placed on the top surface of the prepared specimen before testing. The surcharge simulated the effect of approximately 225 mm thickness of superimposed construction.

The results are detailed in Table 7.7, and are adequate according to the SHW (17). The results indicate a tendency for CBR values to increase with fineness of grading although this has not been clearly defined by the test results. The variation of CBR values with % passing 75 micron sieve is plotted in Figure 7.20.

TABLE 7.7 California bearing ratio test results at varying gradings

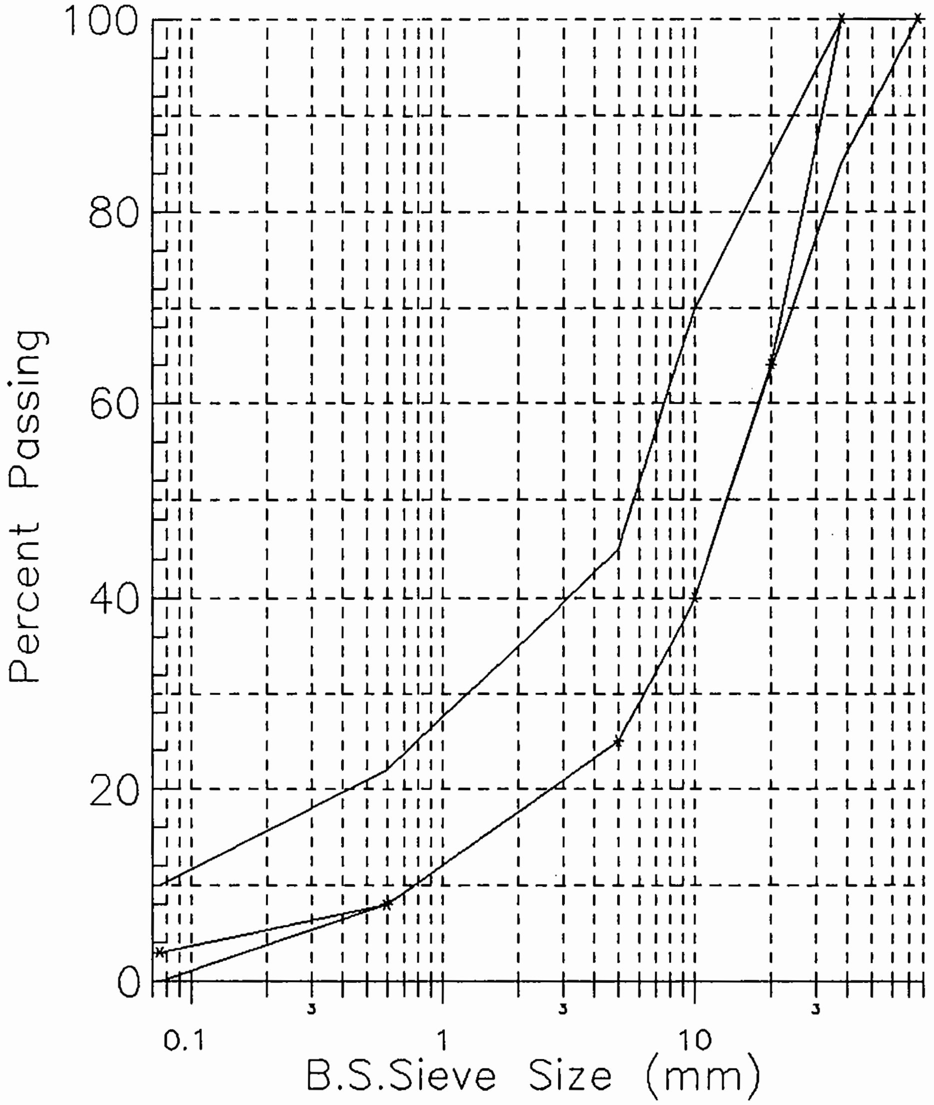
| SAMPLE REFERENCE | CALIFORNIA BEARING RATIO (%) |
|---|---|
| A | 31 |
| B | 33 |
| C | 33 |
| D | 37 |
| E | 35 |
| F | 37 |
| G | 37 |
| H | 40 |

## 7.7 BULK DENSITY

Bulk density tests were carried out in accordance with BS 812:1975: Part 2 (33) at 4% moisture content. The results obtained are detailed in Table 7.8 The results obtained indicate that density is consistent except when the percentage of material passing 75 micron is less than 6% (sample D in Table 7.8). The variation of bulk density with % passing 75 micron sieve is plotted in Figure 7.21.

TABLE 7.8 Bulk density test results at varying gradings

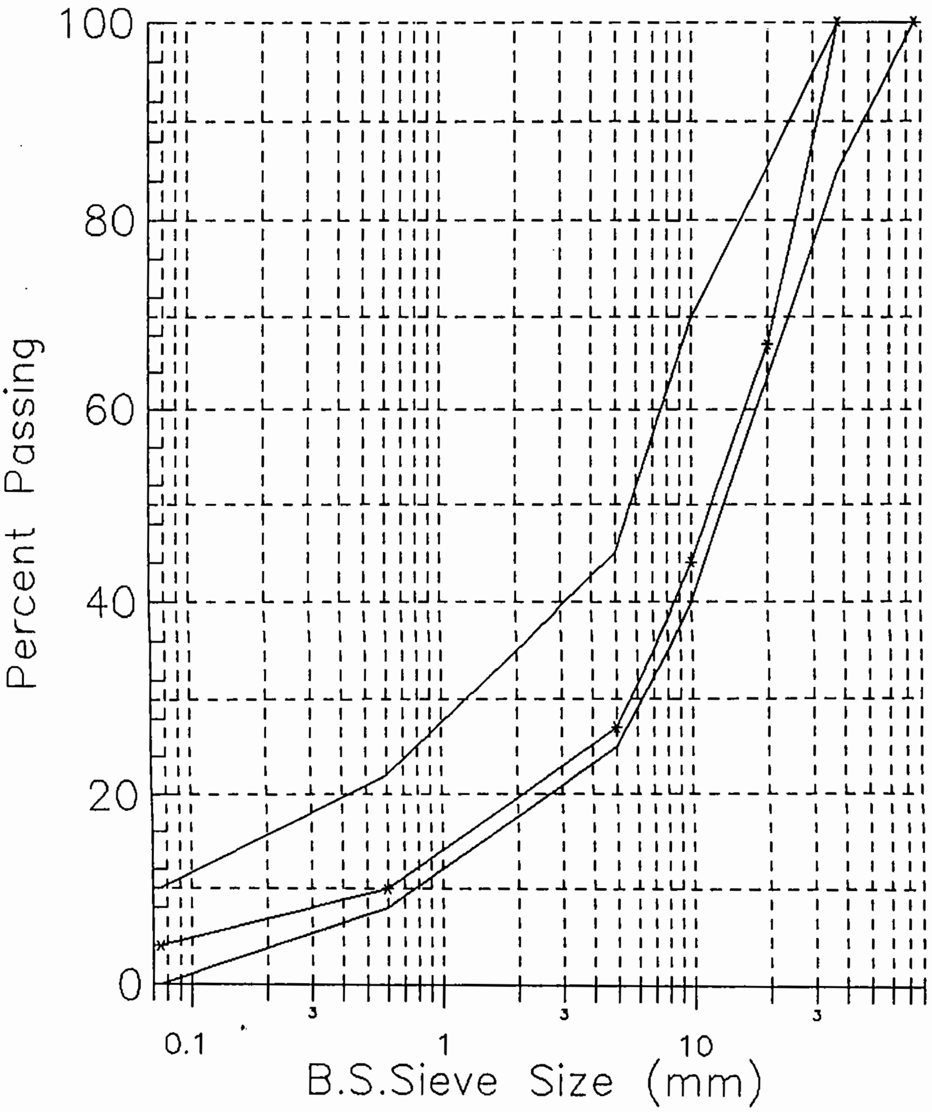
| SAMPLE REFERENCE | COMPACTED BULK DENSITY ($Kg/m^3$) | LOOSE BULK DENSITY ($Kg/m^3$) |
|---|---|---|
| A | 1815 | 1660 |
| B | 1815 | 1680 |
| C | 1830 | 1710 |
| D | 1875 | 1715 |
| E | 1880 | 1715 |
| F | 1870 | 1720 |
| G | 1880 | 1715 |
| H | 1880 | 1720 |




SHW (17) 'Type 1' grading envelope

Sample grading

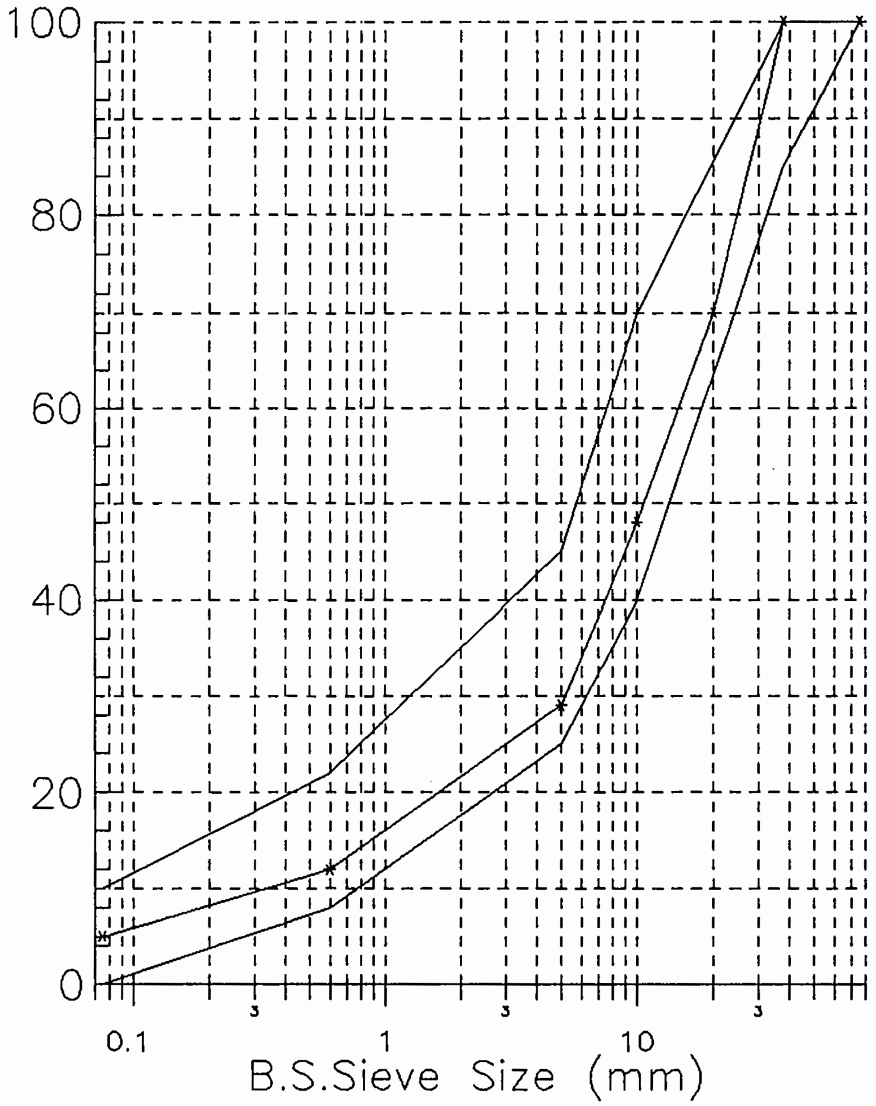
Fig. 7.1 Grading for Sample Reference A




SHW (17) 'Type 1' grading envelope

Sample grading

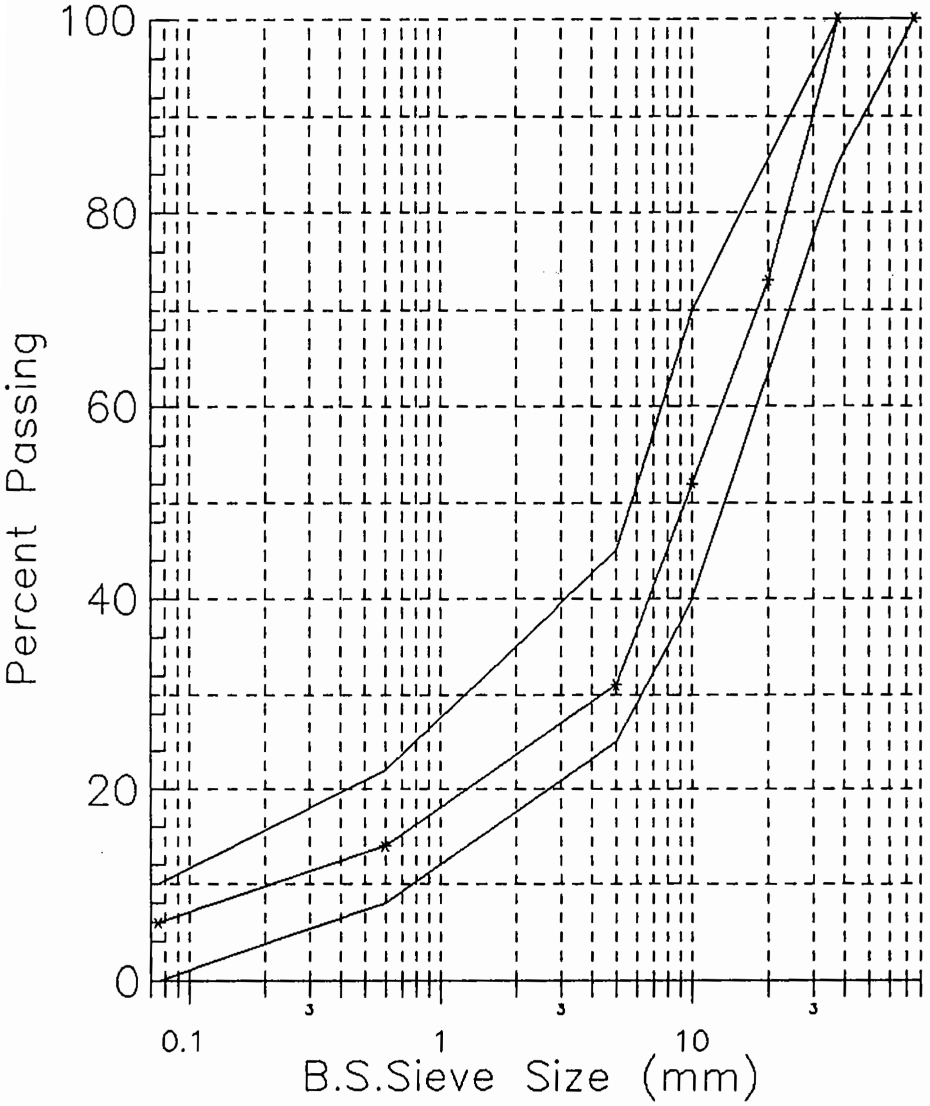
Fig. 7.2 Grading for Sample Reference B




SHW (17) 'Type 1' grading envelope

Sample grading

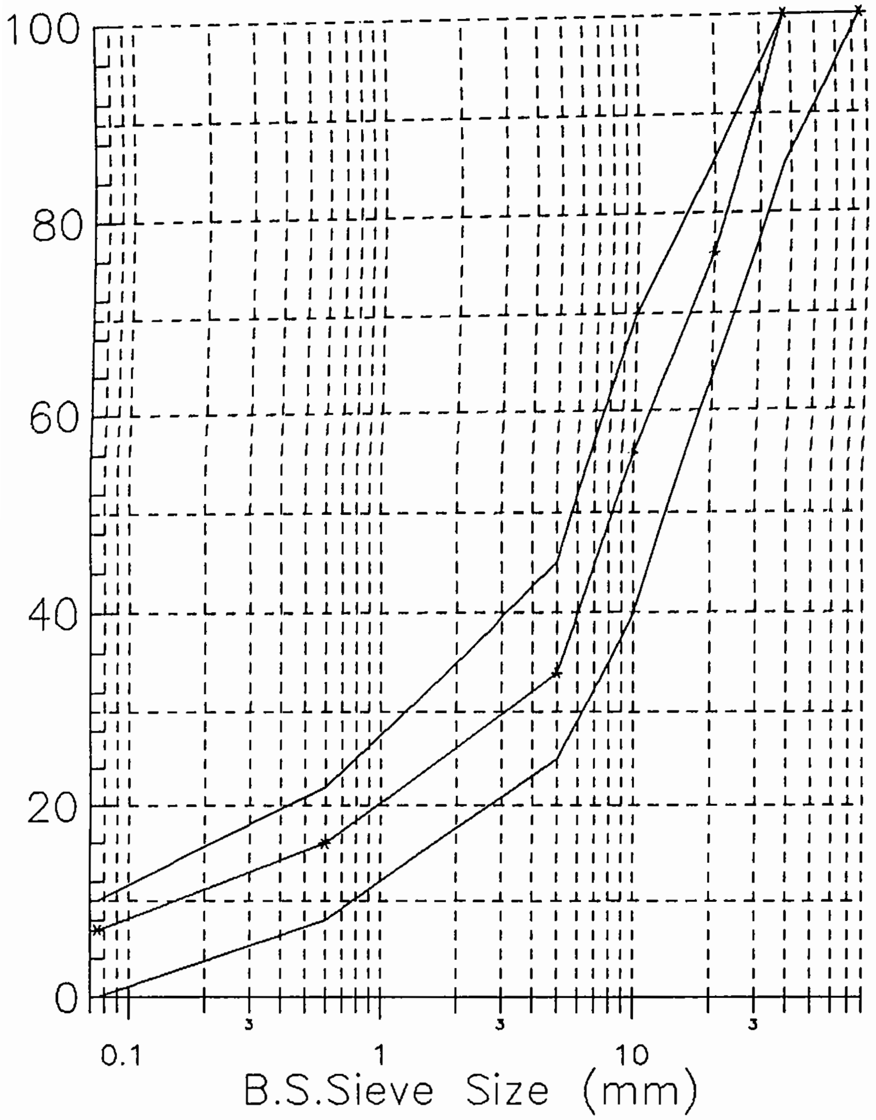
Fig. 7.3 Grading for Sample Reference C




SHW (17) 'Type 1' grading envelope

Sample grading

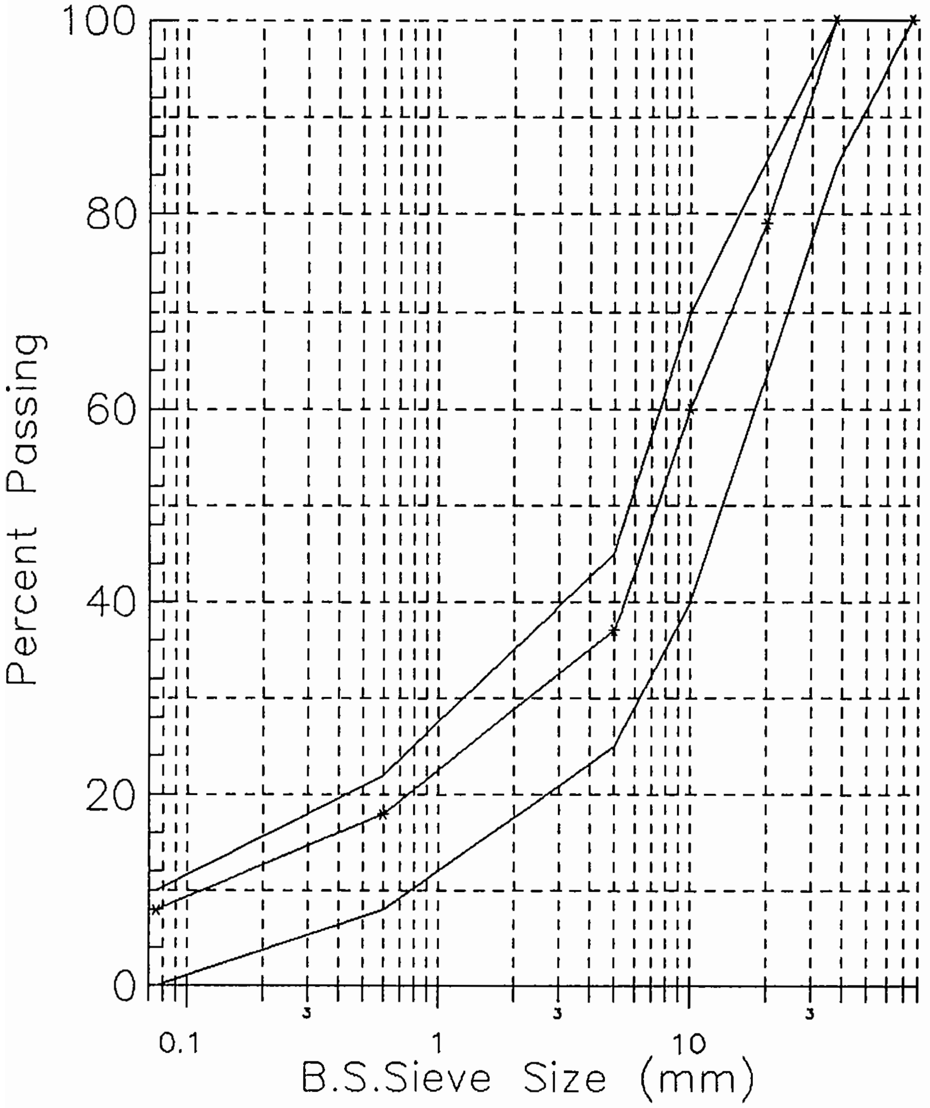
Fig. 7.4 Grading for Sample Reference D




SHW (17) 'Type 1' grading envelope

Sample grading

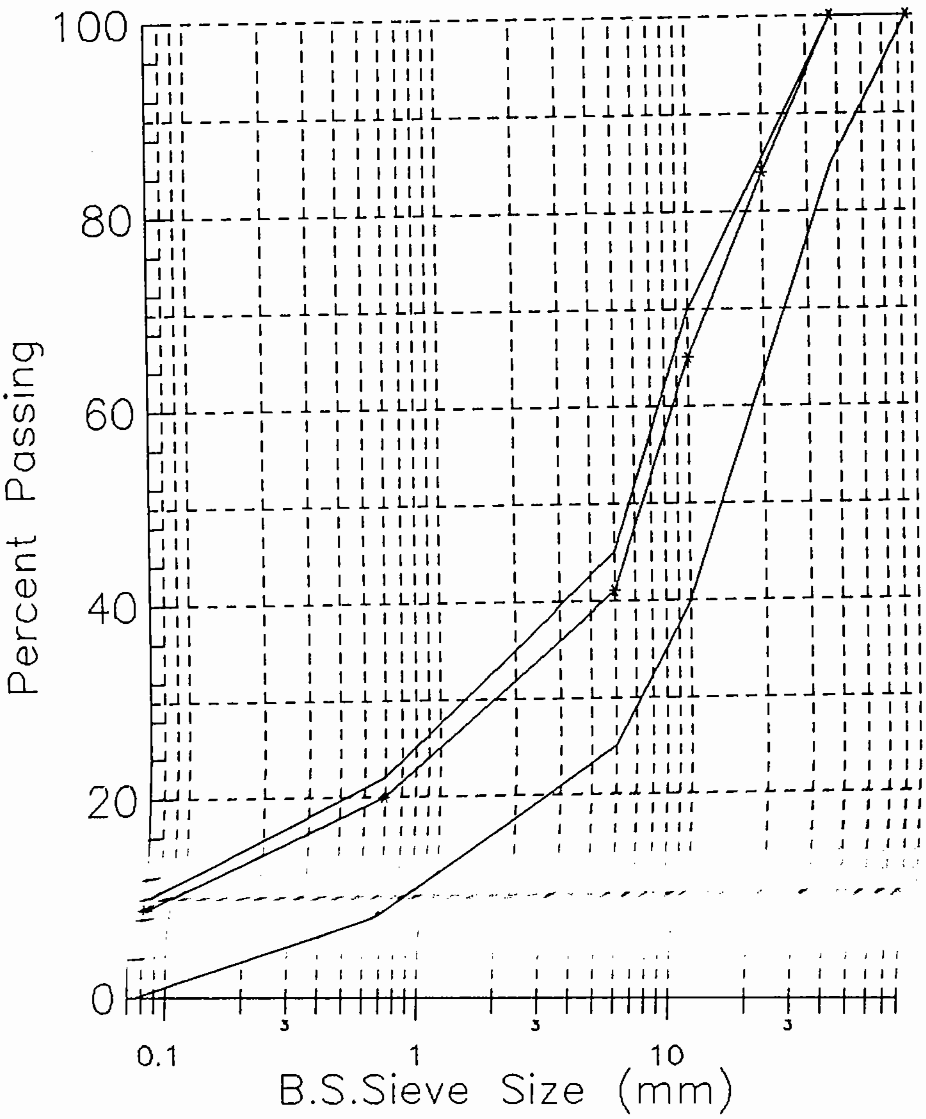
Fig. 7.5 Grading for Sample Reference E




SHW (17) 'Type 1' grading envelope

Sample grading

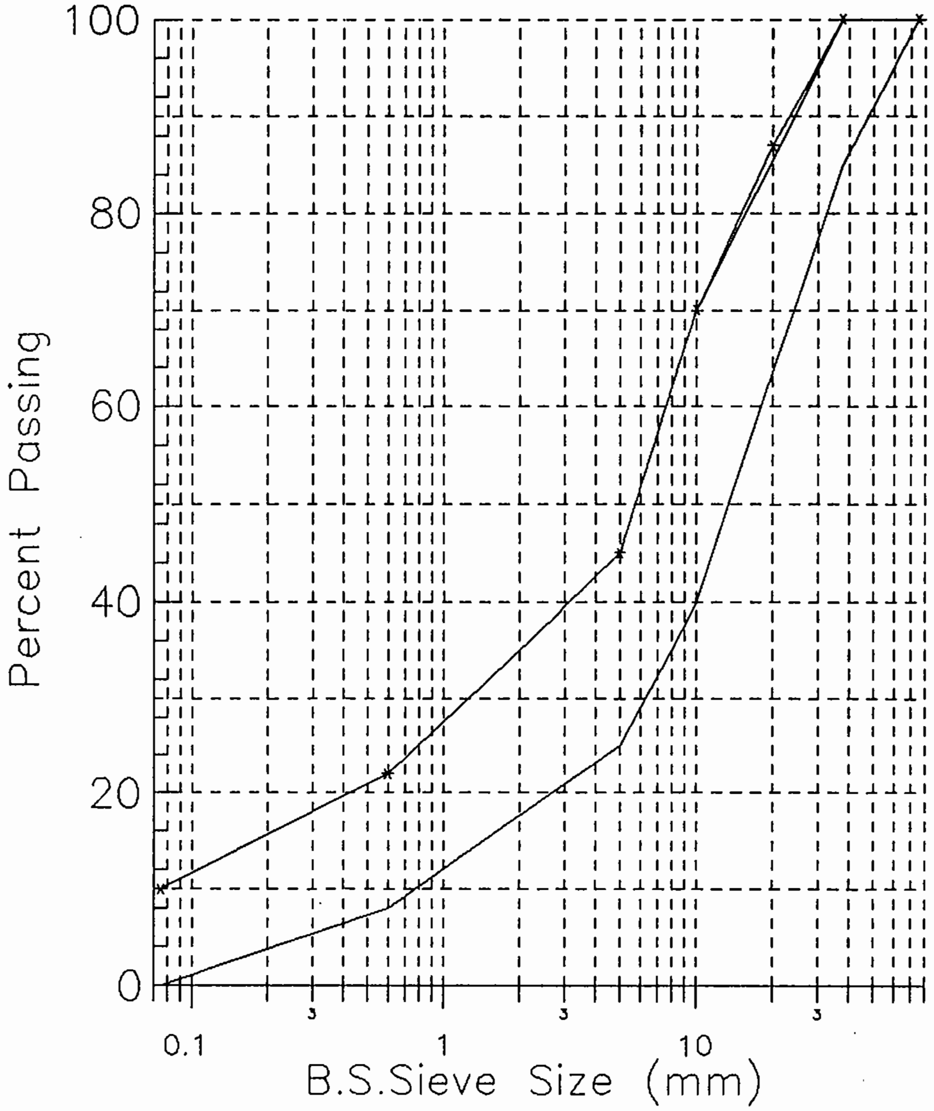
Fig. 7.6 Grading for Sample Reference F




SHW (17) 'Type 1' grading envelope

Sample grading

Fig. 7.7 Grading for Sample Reference G




SHW (17) 'Type 1' grading envelope

Sample grading

Fig. 7.8 Grading for Sample Reference H

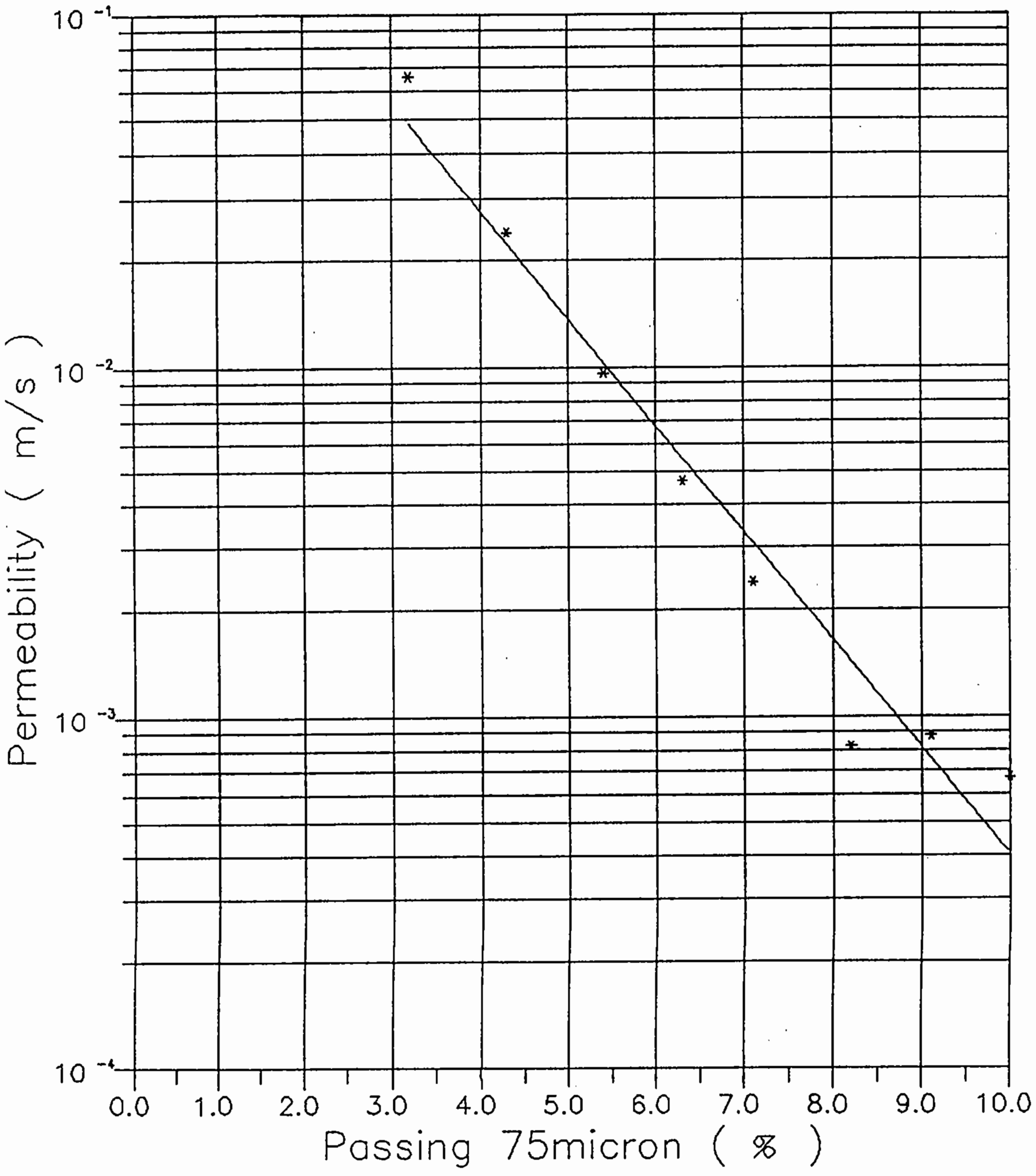



Fig. 7.9 Variation of permeability with percentage (%) passing 75 micron sieve

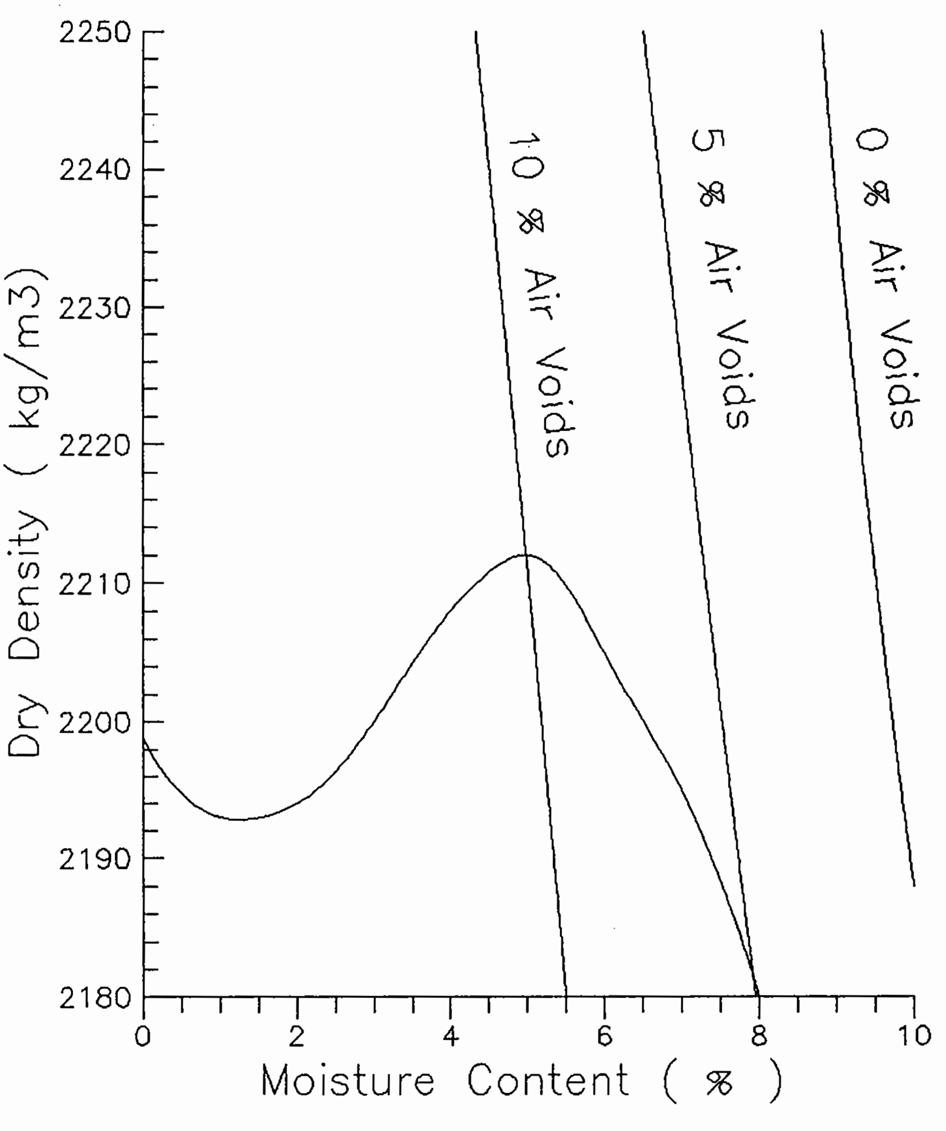



Fig. 7.10 Moisture - density relationship for Sample Reference A

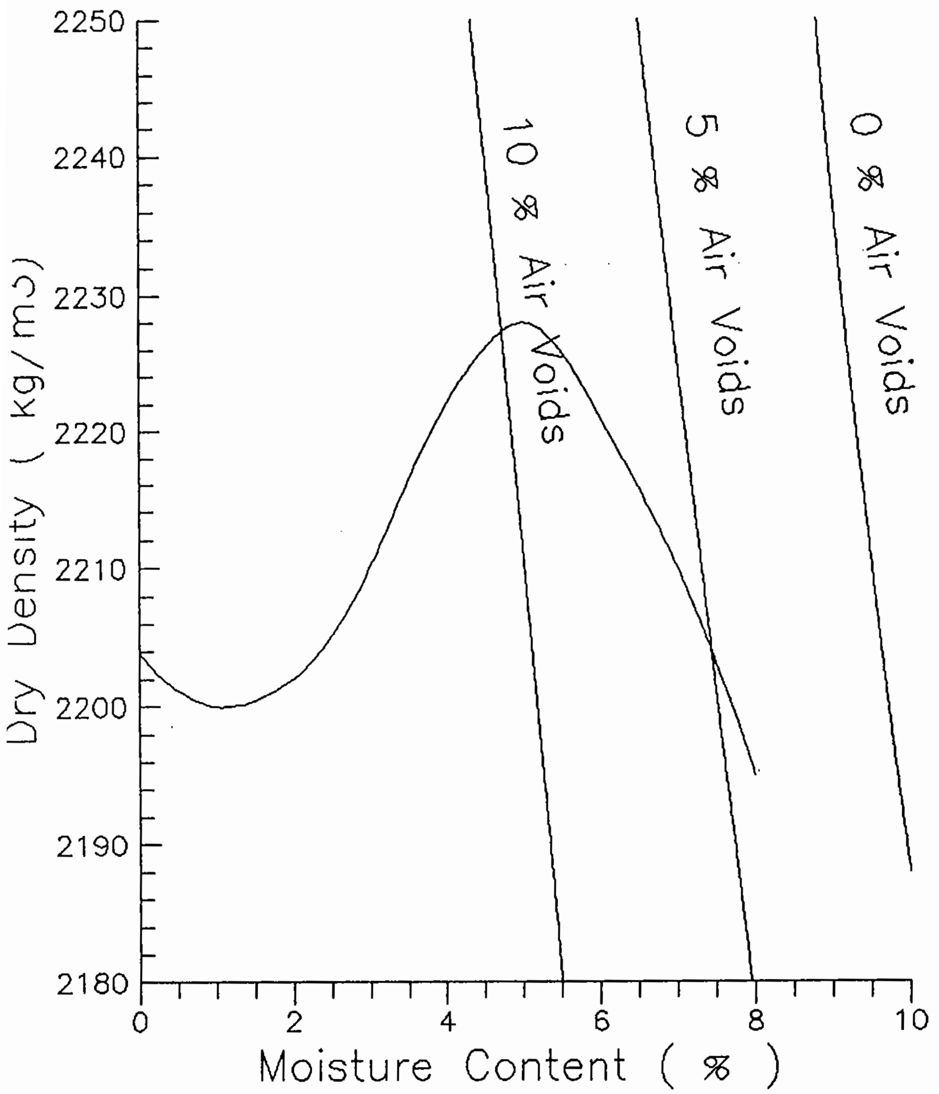



Fig. 7.11 Moisture - density relationship for Sample Reference B

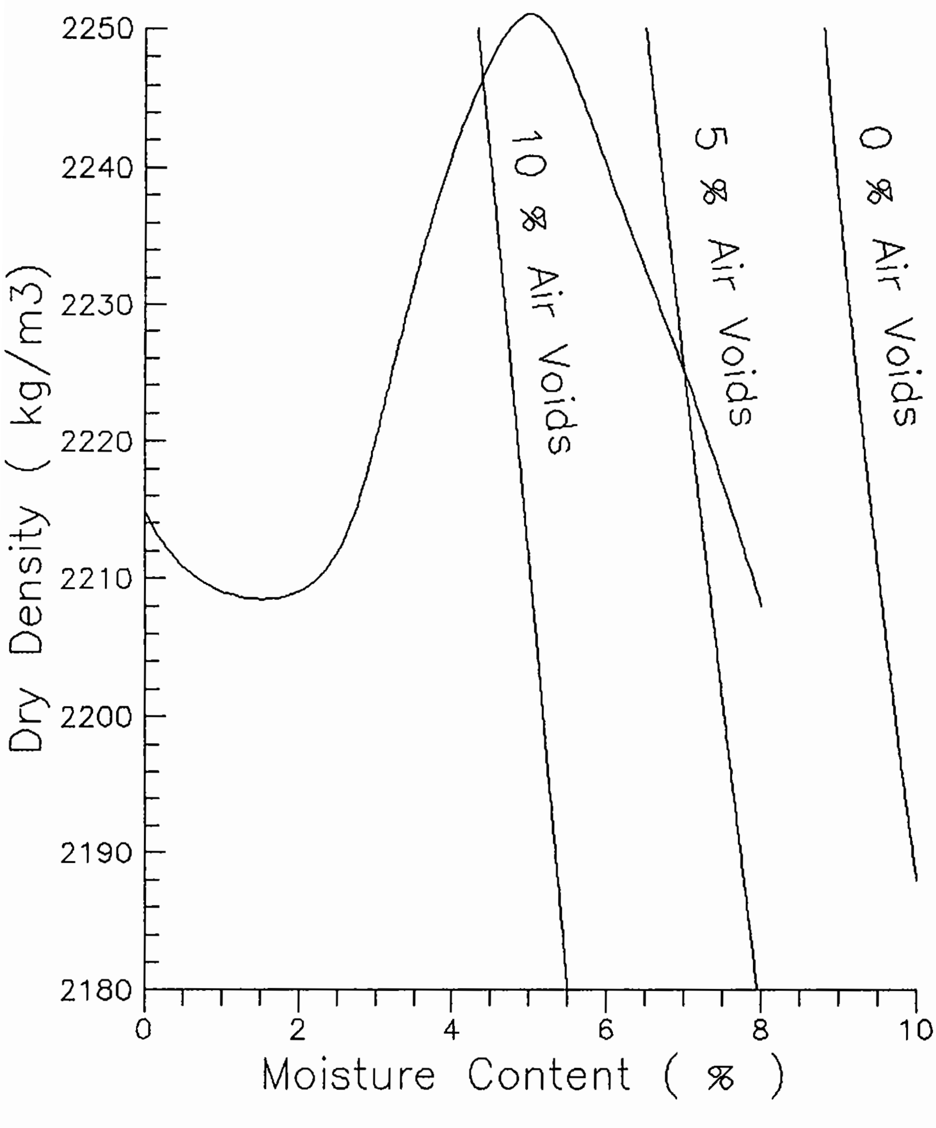



Fig. 7.12 Moisture - density relationship for Sample Reference C

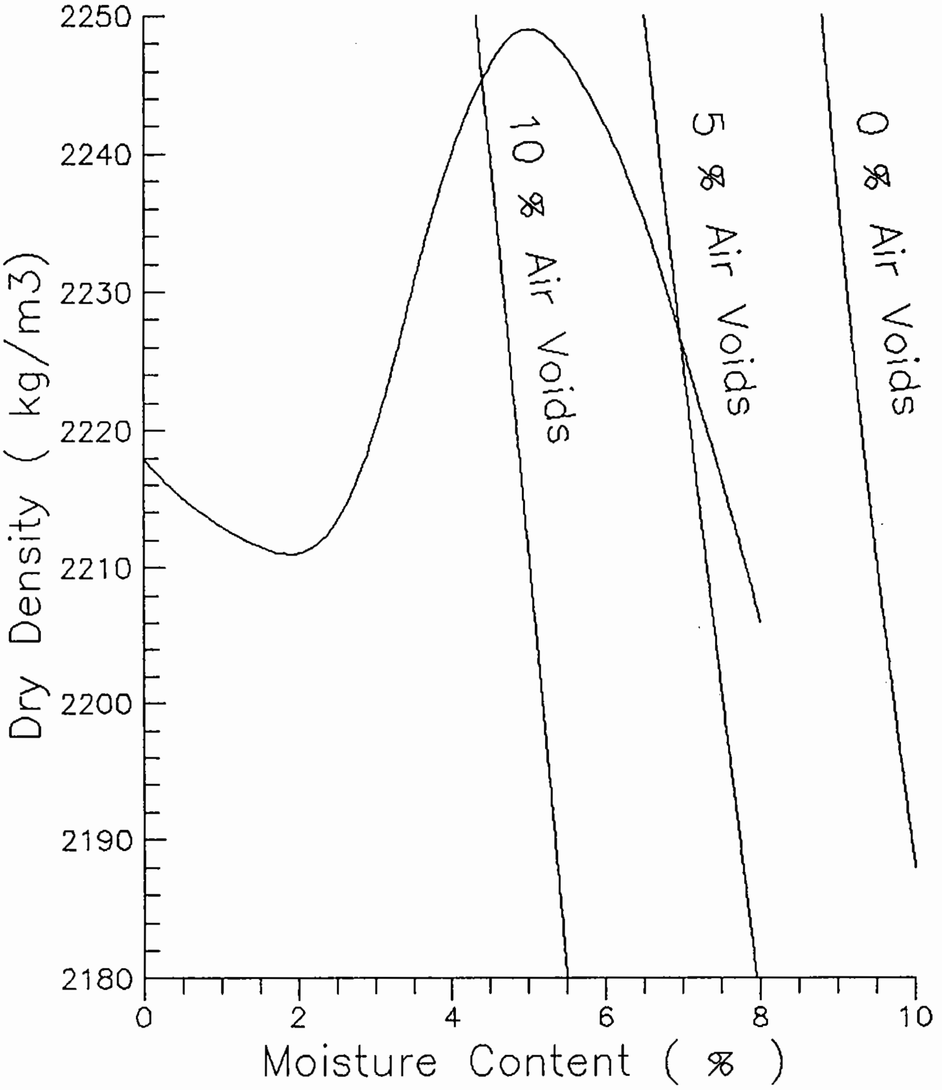



Fig. 7.13 Moisture - density relationship for Sample Reference D

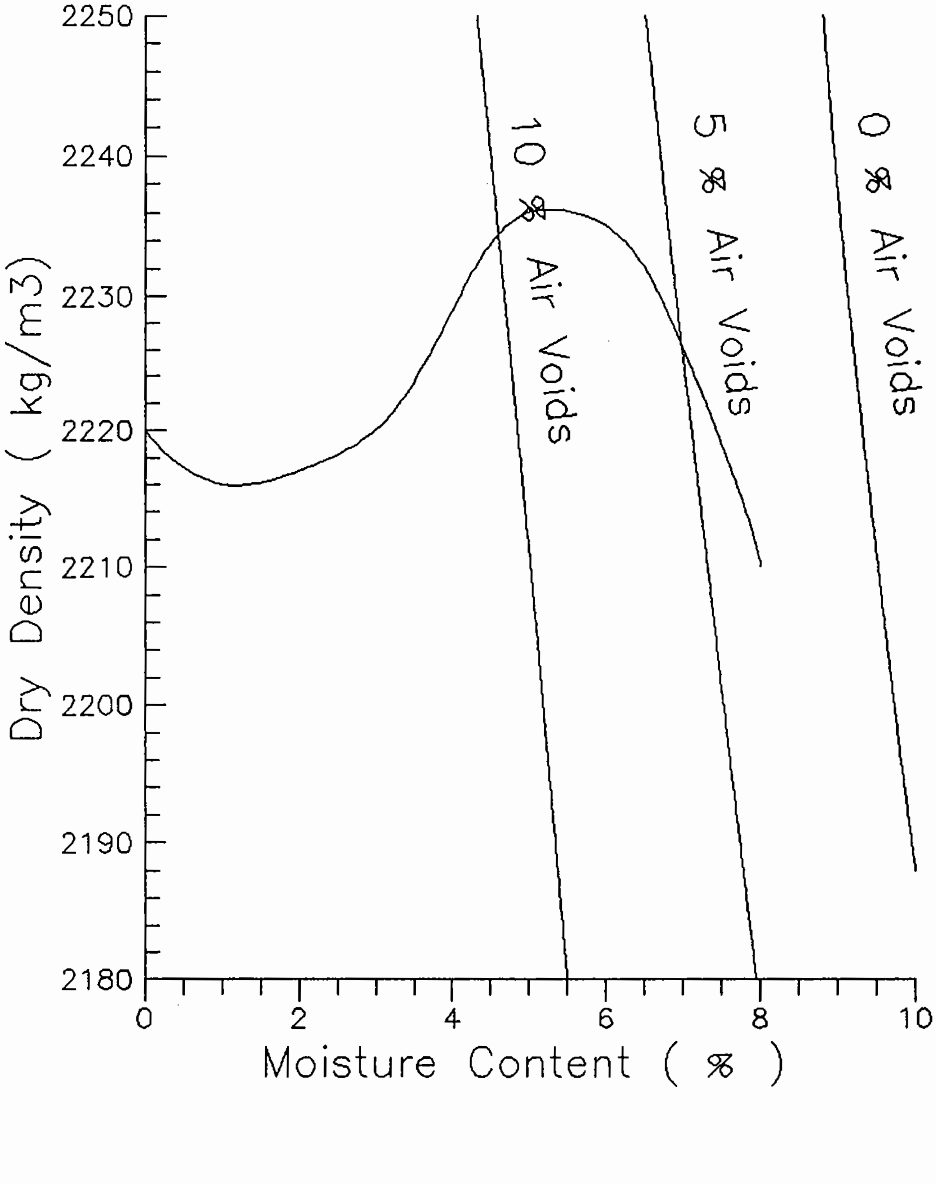



Fig. 7.14 Moisture - density relationship for Sample Reference E

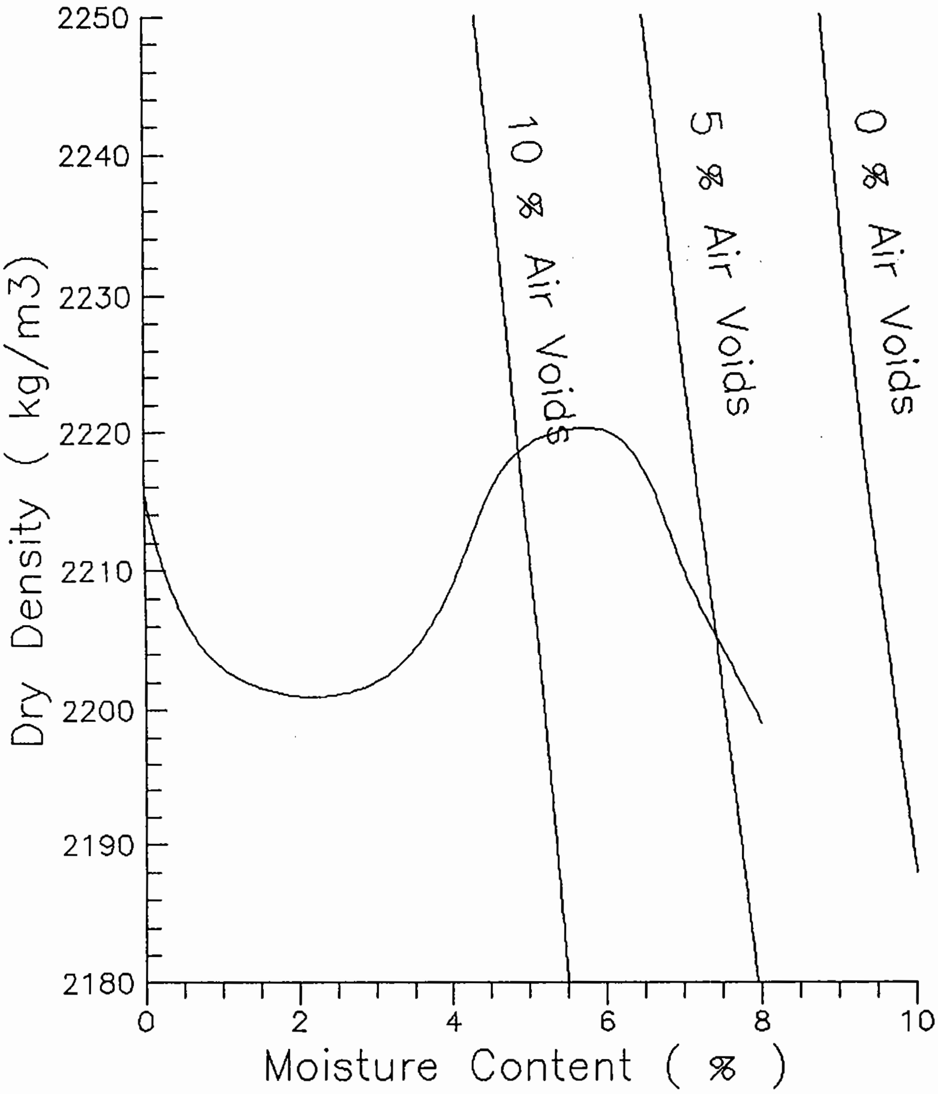



Fig. 7.15 Moisture - density relationship for Sample Reference F

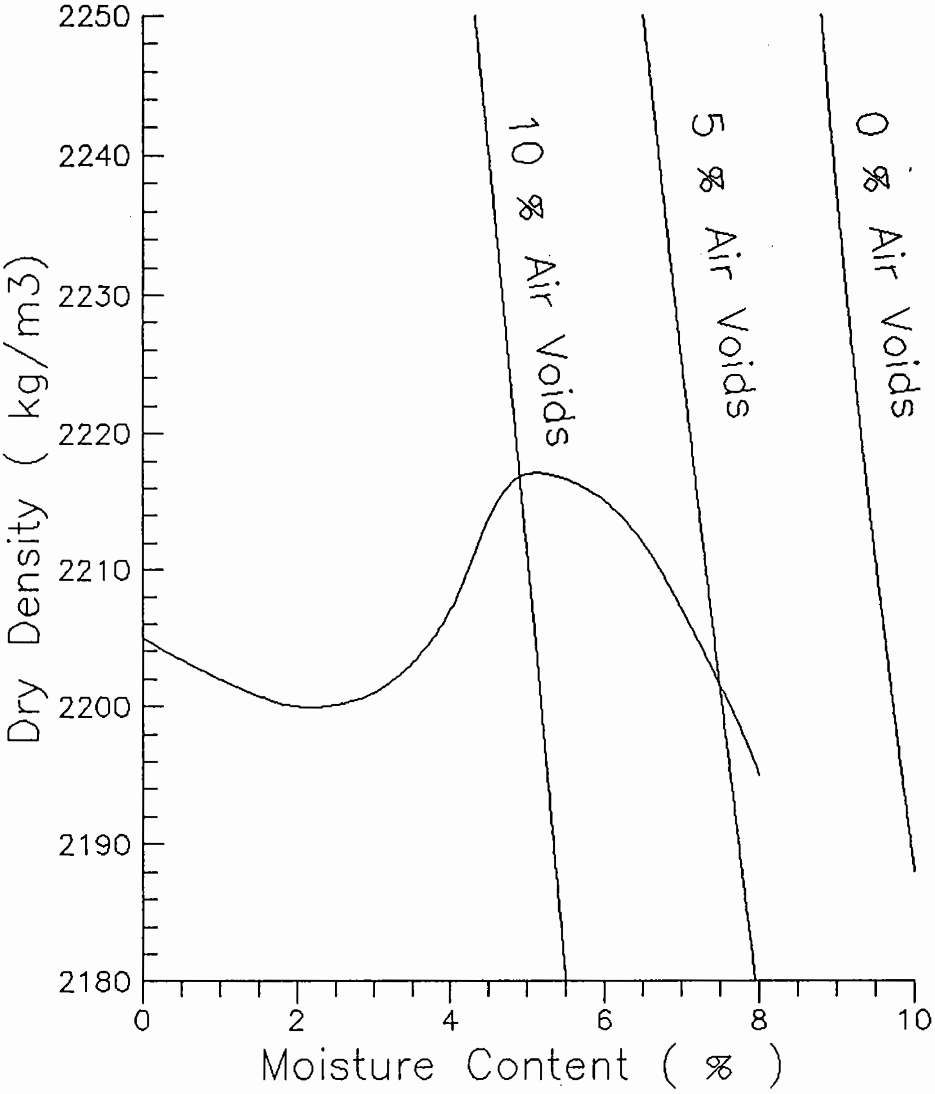



Fig. 7.16 Moisture - density relationship for Sample Reference G

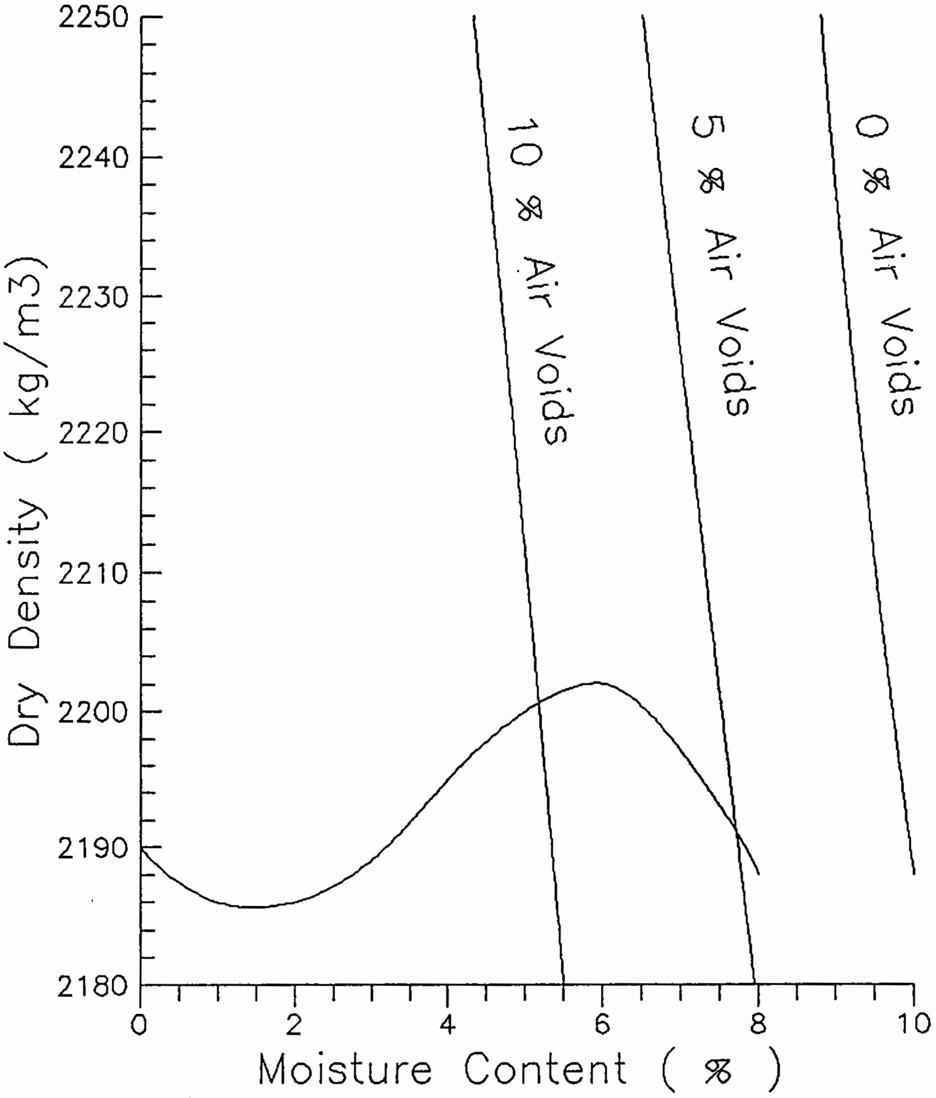



Fig. 7.17 Moisture - density relationship
for Sample Reference H

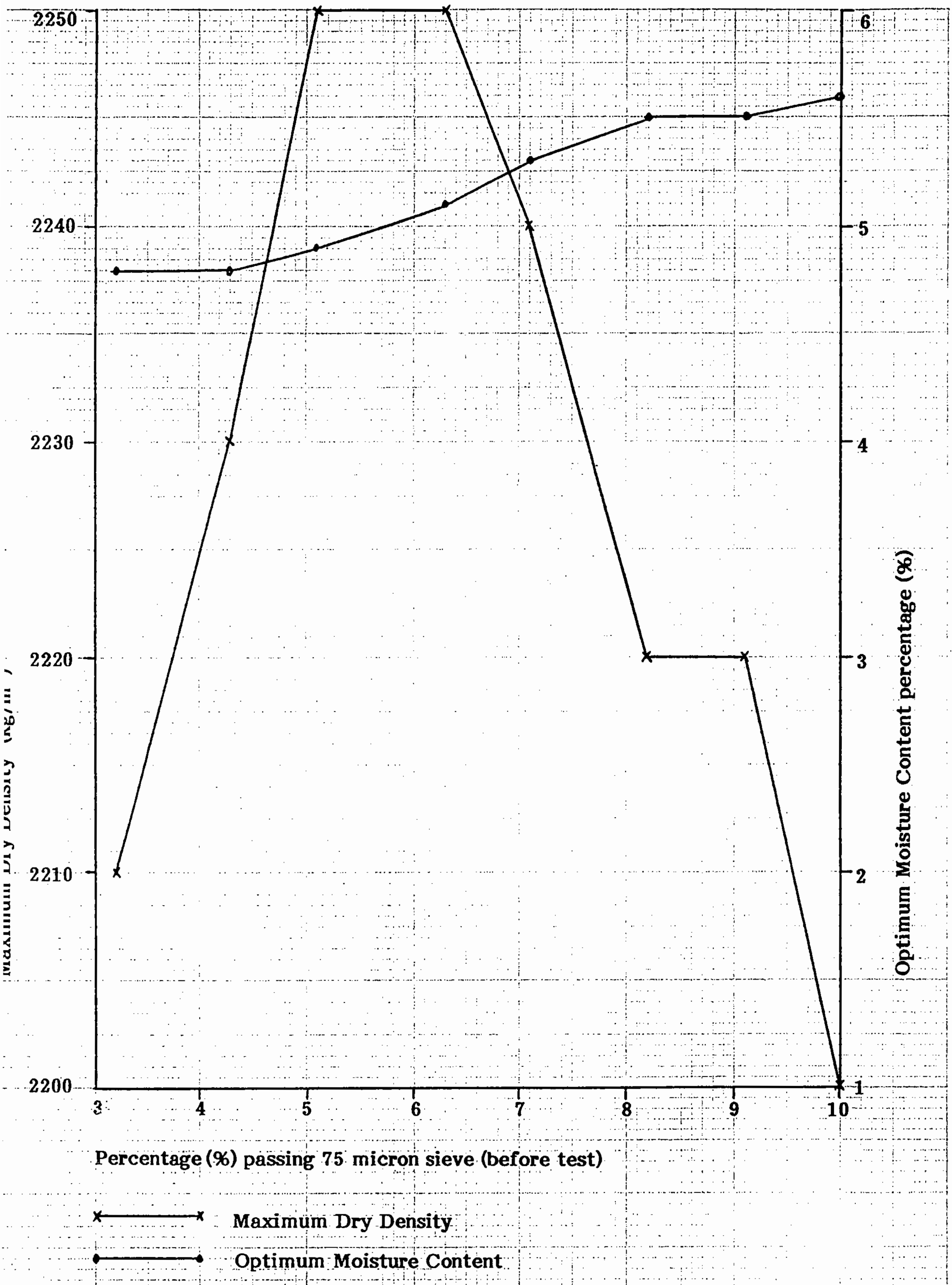



Fig. 7.18 Variation of maximum dry density and optimum moisture content with percentage (%) passing 75 micron sieve

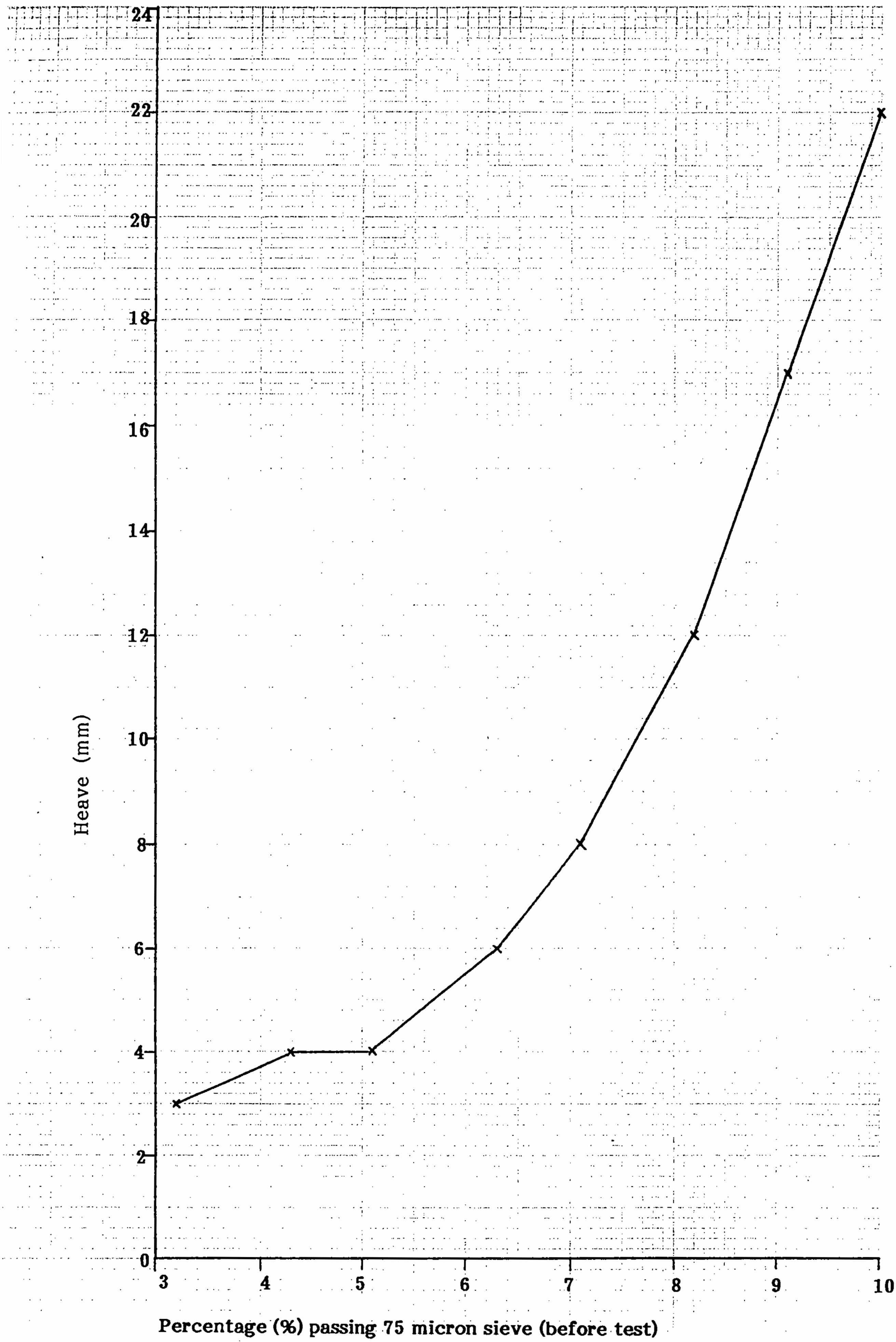



Fig 7.19 Variation of Frost Heave with percentage (%) passing 75 micron sieve

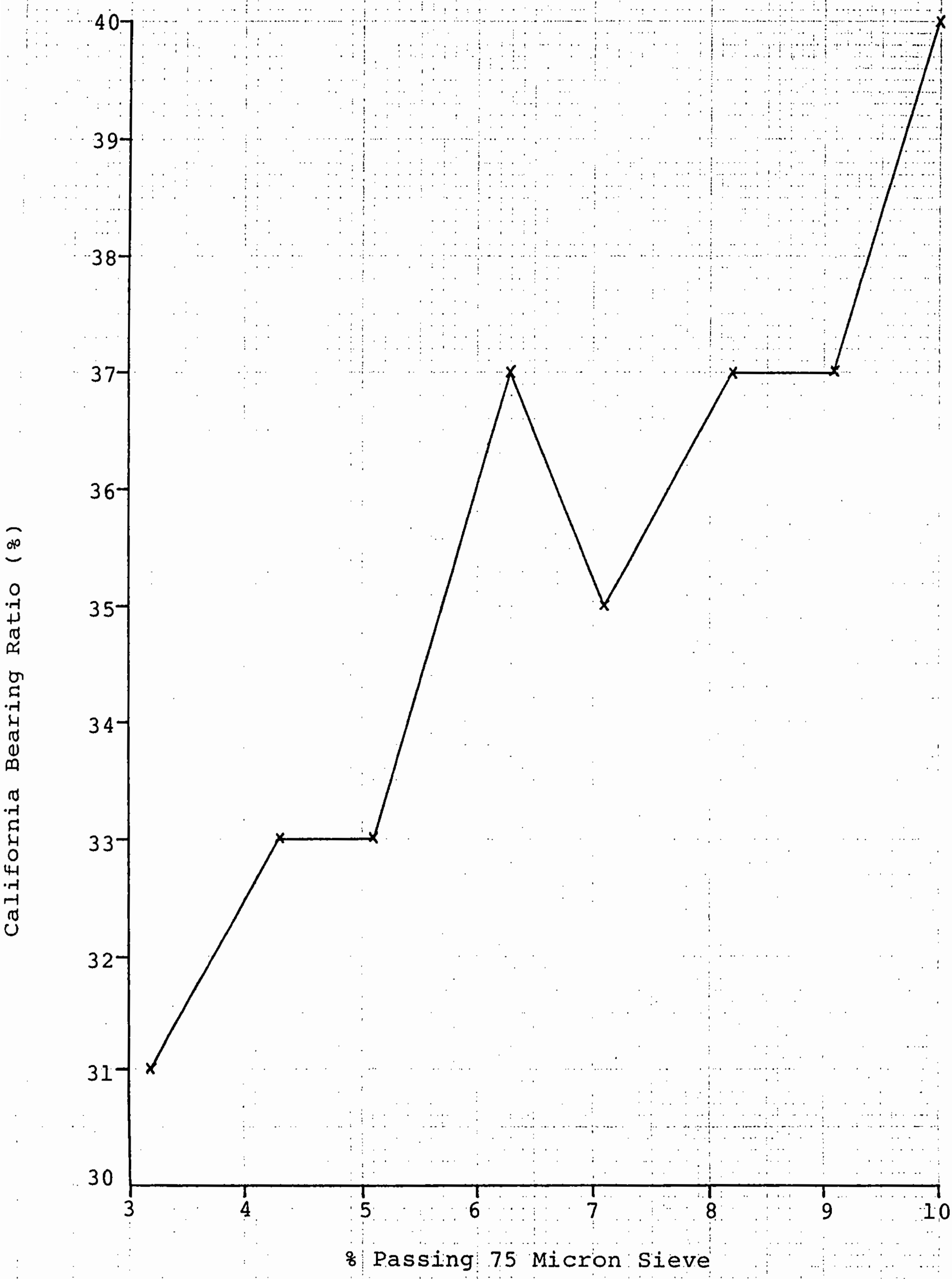



**Fig. 7.20 Variation of CBR values with percentage (%) passing 75 micron sieve**

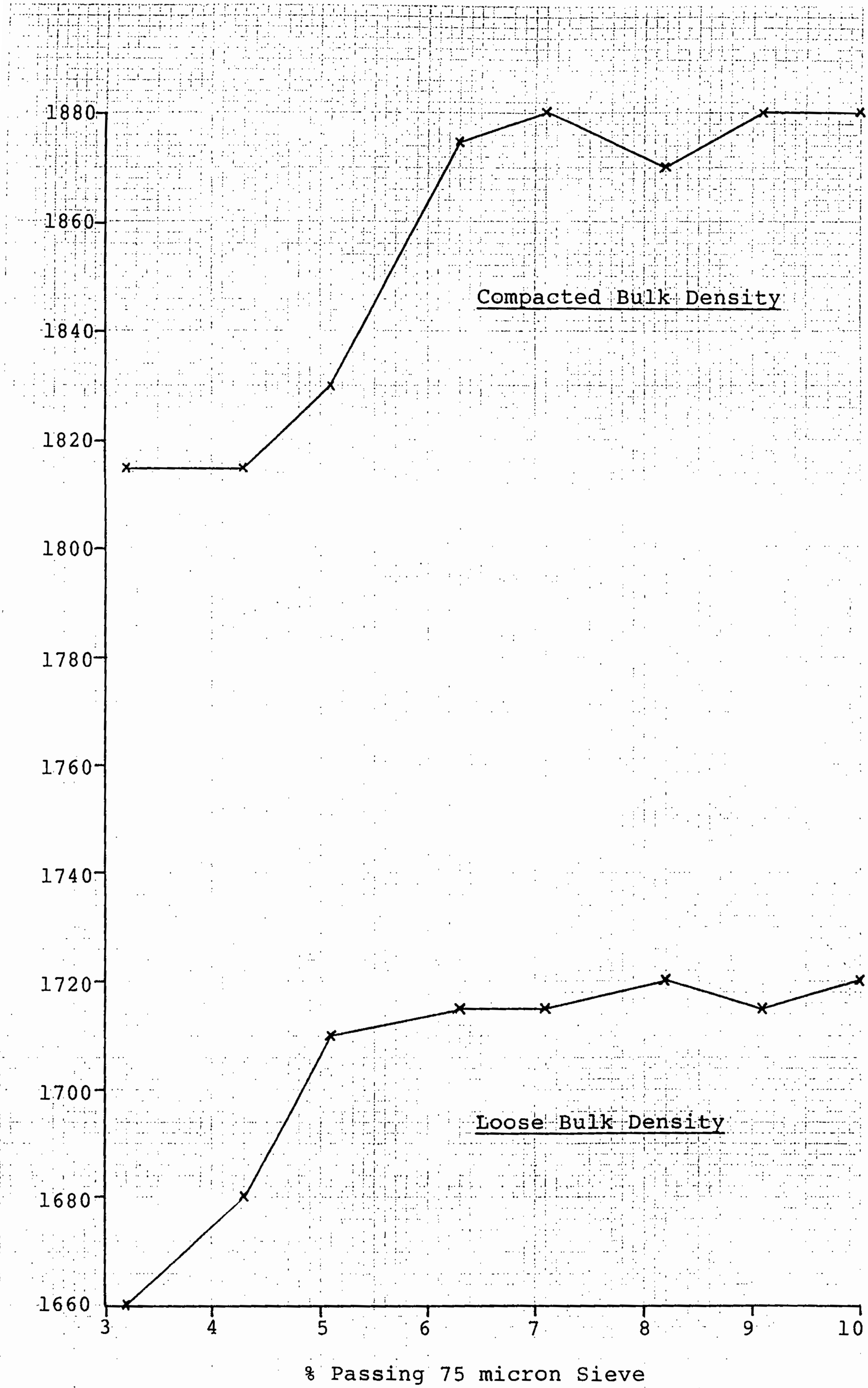



Fig. 7.21 Variation of Bulk Density with percentage (%) passing 75 micron sieve

# 8 CASE STUDY -- BANGOR BY-PASS

## 8.1 GENERAL

Slate waste has been used as an aggregate in sub-bases on several North Wales roads. One of these is the Bangor By-Pass which carries fairly heavy loads due to the container traffic through the nearby port of Holyhead. Performance of the pavement has been variable. Records of this and the associated reconstruction work provided an opportunity for the in-situ performance of slate aggregate to be assessed.

The Bangor By-Pass was constructed between 1980-1982 by Norwest Holst Civil Engineering Ltd. Consultants for the project were Husband & Co. on behalf of the Welsh Office. Materials Consultants employed by the Welsh Office were Nicholls Colton & Partners.

The pavement was founded on a firm clay (CBR >15%) over most of its length with two short sections constructed on concrete bridge decks. The construction of the pavement was 200 mm slate sub-base Type 1, 200 mm limestone wet mix macadam with 100 mm dense bituminous basecourse macadam and 40 mm hot rolled asphalt wearing course. The traffic loading was predicted to be in the range 7-11 msa.

Due to the upgrading of the A55 Coast Road container traffic to Holyhead Port increased substantially soon after the Bangor By-Pass opened. This lead to premature failure due to the inadequacy of the wet-mix macadam and a decision was taken to reconstruct part of the road.

## 8.2 PLATE BEARING TESTING

A series of plate bearing tests were carried out in October, 1989 on the slate sub-base by the method recommended by J. B. A. Day of Lancashire County Council (47) (see Fig 8.1). The tests were carried out using a 300 mm diameter steel plate. A load was applied to the plate which caused deformation in the slate sub-base material. The load was maintained at each increment and settlement measured and recorded at one minute intervals using dial gauges until settlement ceased.

The modulus of sub-grade reaction (k) was then calculated from the formula $k = P/1.3$, where P is the load intensity in $kN/m^2$ mm deflection, and the CBR relationship derived from standard graphs.

The graphs of load versus settlement are plotted in Appendix D and show a modulus of sub-grade reaction varying from 378 to 646 $kN/m^2$ indicating that the slate sub-base was in good condition and had not been responsible

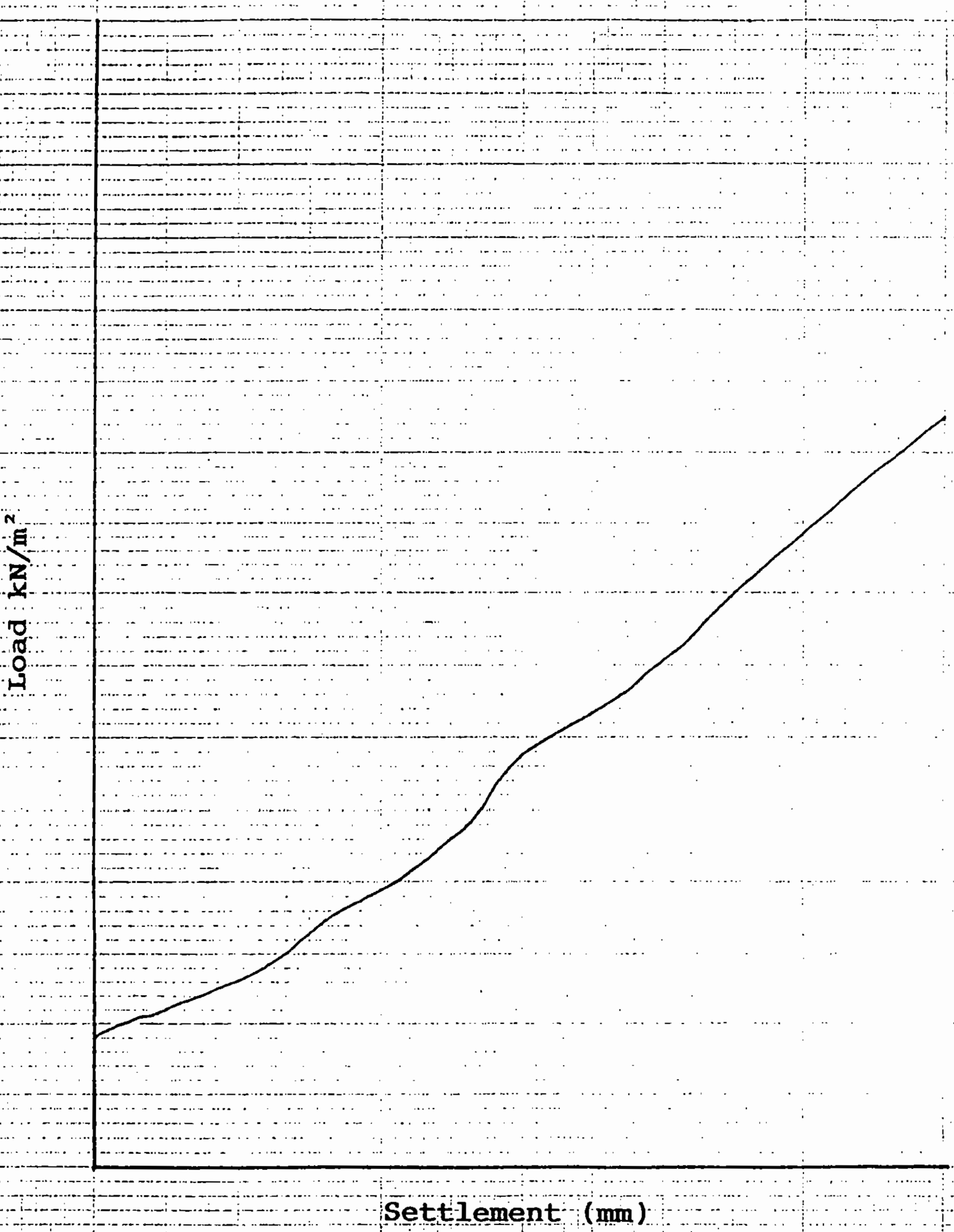



Fig. 8.1 Typical load V settlement plot from Plate Bearing Test

for the premature road failure. Comparative tests carried out on limestone sub-base indicated a range of values from 350 to 540 $kN/m^2$.

Based on these and other test results which indicated that the slate sub-base was in good condition it was decided to reconstruct the road with an additional 100 mm layer of slate sub-base with a 100 mm layer of 40 mm heavy duty road base, a 100 mm layer of 20 mm dense bituminous basecourse and 40 mm of hot rolled asphalt wearing course.

## 8.3 DEGRADATION

The original grading results of the slate sub-base were available and ten samples of uncovered slate sub-base were taken from site in order to assess the amount of degradation that may have taken place.

Grading results are tabulated and plotted in Appendix E. The mean change in grading of the ten samples is given in Table 8.1.

TABLE 8.1 Change in grading of Type 1 sub-base material after approximately 7 years of traffic

| BS SIEVE SIZE | MEAN OF ORIGINAL GRADINGS (% PASSING) | MEAN OF 10 UNCOVERED GRADINGS (% PASSING) | CHANGE IN GRADING |
|---|---|---|---|
| 75.0 mm | 100 | 100 | 0 |
| 37.5 mm | 98 | 99 | + 1 |
| 20.0 mm | 75 | 79 | + 4 |
| 10.0 mm | 55 | 61 | + 6 |
| 5.0 mm | 31 | 32 | + 1 |
| 600 micron | 10 | 11 | + 1 |
| 75 micron | 3.9 | 4.2 | + 0.3 |

Bearing in mind that the original gradings of the material available represented 100 tonnes of Type 1 sub-base sampled at the time of delivery to site before placing, and that records of chainages of original samples do not identify in which lane of the carriageway the material was laid, and that the uncovered areas each represented no more than 1 $m^2$ of material, it is difficult to apply a high degree of accuracy to the correlation. In view of this it is surprising that the correlation between the original and recovered gradings are so close. The results do however suggest that a small percentage degradation is occurring overall which is more evident at the 20 mm and 10 mm fractions. This degree of degradation would be expected even with conventional aggregates.

The most intense loading of a sub-base material occurs during construction. In order to assess degradation under site traffic it was decided to carry out trials by trafficking compacted sub-base with construction plant.

A 4.5 metre wide, 50 metre long strip of new slate sub-base was laid approximately 100 mm thick and compacted with 6 passes of a Bomag 200 twin drum vibrating roller. The mass per metre width of the roller was
1850 Kg. The material was laid at the supplied moisture content which varied between 3.6% and 4.1%. No segregation was observed during laying and compacting.

During the compaction process samples of material were taken for grading analysis after 1, 2, 4 and 6 passes of the roller. The samples taken were approximately 15 Kg, the mass being kept low to avoid excessive disturbance of the layer. This lead to a greater scatter of results than desirable. The density of the material was checked at each sample location using a Troxler 3411B Nuclear Density Gauge in the direct transmission mode. Preliminary trials with the machine in backscatter mode indicated that the correlation between the measured density and actual density was not as good as when the machine was in direct transmission mode. A 20 mm diameter hole was formed in the sub-base by driving a hardened steel pin to a depth of 100 mm and the gauge set to record in direct transmission at 50 mm depth. No levelling medium was found necessary. For each location, four measurements were taken at the four points of the compass. A number of additional locations were also checked for density in the same manner after each pass of the roller.

The hole left after sampling was filled with new material and no further samples were taken from that position. The results of the grading analyses and density tests are detailed in Appendix F. A comparison of the grading analyses after laying and after six passes of the roller is given in Table 8.2 and shown in Figure 8.2.

The grading analysis results indicate that a small breakdown of the aggregate particles has occurred during compaction. The results of this trial tends to confirm that the apparent change in grading recorded in Table 8.1 is a genuine feature.

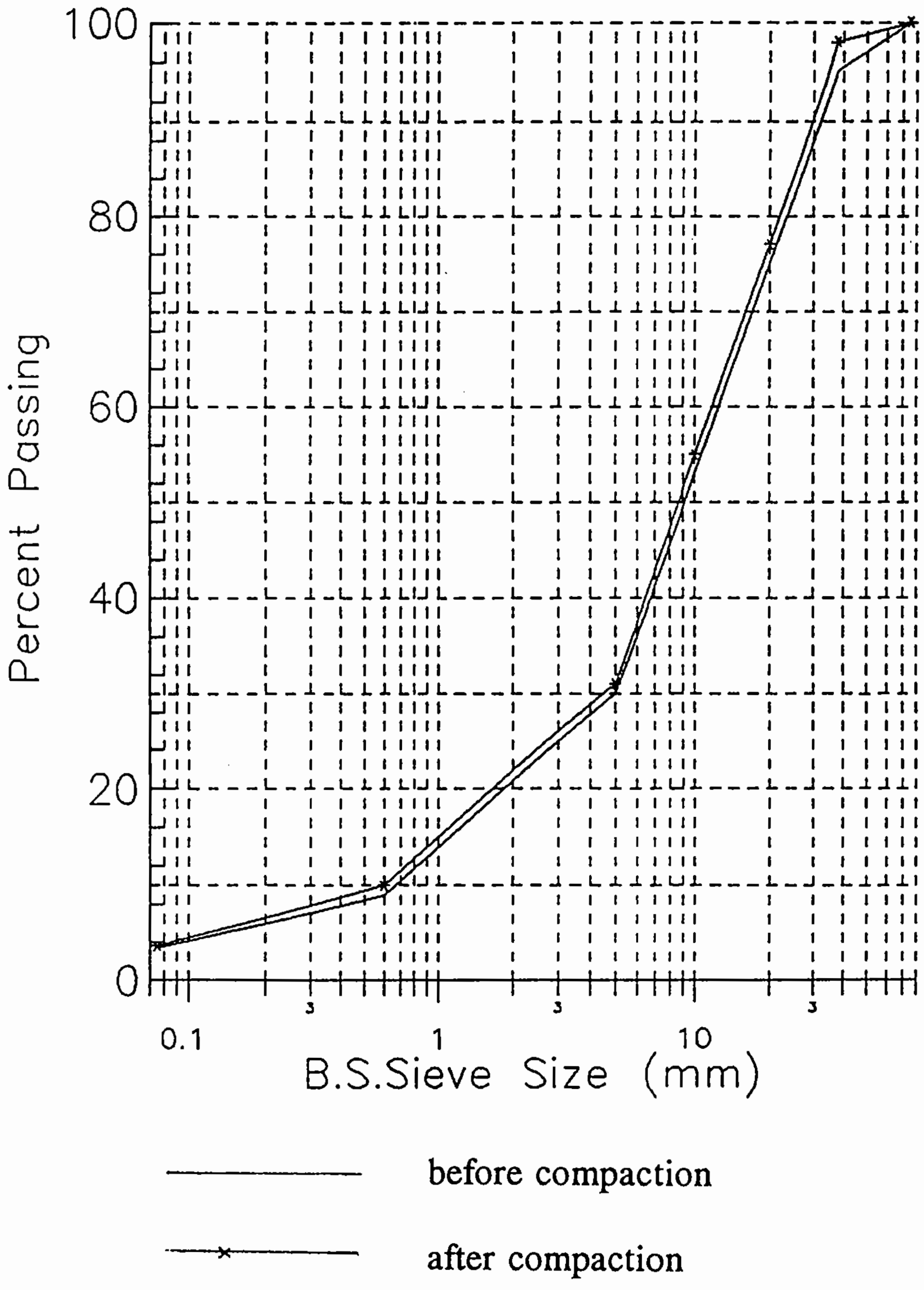



Fig. 8.2 Comparison of grading analyses after six passes with vibratory roller

TABLE 8.2 Comparison of grading analyses after six passes with vibratory roller

| Sample No. | K | | L | | M | | N | | Mean | | Mean Change |
|---|---|---|---|---|---|---|---|---|---|---|---|
| No of Roller Passes | 0 | 6 | 0 | 6 | 0 | 6 | 0 | 6 | 0 | 6 | |
| BS SIEVE SIZE | PERCENTAGE PASSING | | | | | | | | | | |
| 75.0 mm | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 0 |
| 37.5 mm | 98 | 100 | 100 | 100 | 92 | 96 | 91 | 96 | 95 | 98 | +3 |
| 20.0 mm | 76 | 78 | 80 | 83 | 71 | 72 | 73 | 76 | 75 | 77 | +2 |
| 10.0 mm | 54 | 56 | 59 | 61 | 49 | 52 | 50 | 49 | 53 | 55 | +2 |
| 5.0 mm | 30 | 31 | 34 | 35 | 27 | 29 | 28 | 30 | 30 | 31 | +1 |
| 600 micron | 9 | 10 | 10 | 11 | 9 | 9 | 9 | 10 | 9 | 10 | +1 |
| 75 micron | 3.4 | 3.7 | 3.9 | 4.0 | 3.1 | 3.4 | 3.0 | 3.4 | 3.4 | 3.6 | +0.2 |

The density of the material after six passes of the roller indicate that full compaction had taken place as there was at most a 0.4% increase in density between four and six passes. The results of density at six passes correlates well with the maximum dry density results achieved in the laboratory using the vibrating hammer test (see Table 7.4). These density trials suggest that the material is easy to compact and that specified densities of a minimum 95% of maximum dry density should be readily achievable.

## 8.4 COMPACTION

The density of a compacted lift decreases from the surface downward in proportion to the increase in pressure intensity except for the loose surface due to vibration. Pressure intensity from a surface load decreases rapidly downward. Thus to obtain appropriate compaction of a layer of granular material, a compactor that yields a high intensity surface load must be employed. This however may have the disadvantage of causing excessive sinkage, rutting and pushing of the material ahead of the roller. These problems were not evident with the slate sub-base.

Trial areas of sub-base were tested on the Bangor By-Pass using various types of compaction equipment - smooth wheeled, grid and vibratory rollers. The results of the densities achieved are detailed in Table 8.3.

TABLE 8.3 Compacted density using various rollers

| ROLLER TYPE | LIFT THICKNESS (mm) | MEAN DRY DENSITY (Kg/m$^3$) |
|---|---|---|
| SMOOTH WHEELED | 100<br>150 | 2200<br>2170 |
| GRID | 100<br>150 | 2210<br>2190 |
| VIBRATORY | 100<br>150 | 2250<br>2240 |

Roller Details:-

| | | |
|---|---|---|
| Smooth Wheeled | : | 10.4 tonne deadweight |
| Grid | : | 9.2 tonne deadweight |
| Vibratory | : | 1.85 tonne per metre width of roll |

# 9. DISCUSSION

## 9.1 AGGREGATE TESTS

As aggregates obtained from different sources differ considerably in their constitution and properties (see Table 6.3) inevitably they differ also with regard to their engineering properties. Metamorphic rocks of the foliated varieties, owing to the preferential orientation of flaky minerals (slaty, schistose and gneissose textures), are comparatively low in toughness and wearing resistance. It is necessary therefore to carry out various tests to determine the engineering properties and to establish the mechanical and physical properties of the material.

### 9.1.1 NON-MECHANICAL QUALITY TESTS

Non-mechanical quality tests are carried out on the aggregate with the intention of determining its suitability for a specific use. The results are normally compared with aggregate specifications to see whether they comply with the desired properties and characteristics. The tests of particular interest are the shape and water absorption tests.

Use of the shape tests in specifications is based on the view that internal friction is influenced by the shape of the particles. The internal friction of an aggregate is the property which by means of the interlocking of particles and the surface friction between adjacent surfaces, restricts particle movement under the action of an imposed load. Thus for instance, the British Standard (48) for single sized aggregates has a particular requirement relating to flakiness since the aggregate cannot rely upon gradation to achieve high density, and consequently, high internal friction.

Flakiness and elongation index limits when specified are generally required to be not greater than 25. The slate aggregates tested are amongst the most flaky of aggregate types. The elongation index values determined in the laboratory are also high, 23-34, in comparison to conventional aggregates 10-25. This property has led contractors to experiment with various types of compacting roller which are intended to reduce the elongation of the particles. It is not known how successful this has been.

The flaky, elongated and smooth properties of slate aggregate have been of great concern to engineers who have used slate aggregate. This concern has led the author to pay particular attention to the possible detrimental effects of the high flakiness and elongation values. The behaviour of the material during compaction was monitored in a trial area as detailed in Section 8.3 and is discussed in

Sections 9.3 and 9.4. There was no evidence of excessive sinkage, rutting and pushing of material ahead of the roller thus indicating that the internal friction of the slate sub-base is adequate irrespective of its flaky and elongated nature.

Knowledge of the water absorption properties of an aggregate is useful when evaluating the results of durability tests carried out on marginal aggregates for use at particular locations, as some of the weaker aggregates would have their test values reduced if they were subjected to testing under saturated conditions. The absorption values obtained on the slate aggregates indicate that the material is amongst the least absorbent of all aggregates. This is hardly surprising considering the use of slate as a roofing material.

In addition to the strength requirements discussed in Section 9.1.2 resistance to a considerable degree of weathering during the service lifetime of an unbound pavement layer is required including resistance to disintegration with cycles of wetting and drying. Some of the severest cases of roadstone disintegration in unbound bases have been associated with physical breakdown of rock which had high water absorption values (49). In the failed areas a layer of saturated high plasticity fines has practically always been found between the surface layers and the underlying sub-base. This slurry is formed when aggregate containing secondary minerals degrades into plastic fines under the combined action of water plus movement of stone against stone (attrition) as the pavement is flexed under moving loads. The plastic fines quickly permeate the entire base course. When this happens the stability of the base course is reduced sharply and the likelihood of pavement structure failure is greatly enhanced.

Sedimentary rocks employed as aggregates, chiefly the sandstones and limestones, being themselves the product of weathering processes suffer comparatively little from mineral alteration. They do however suffer to varying degrees from softening and disintegration due to wetting and drying. Porosity and water absorption are the main factors influencing the susceptibility to these mechanical weathering effects. The properties of low water absorption and non-plastic nature of the fines would appear to give slate aggregate an advantage over sandstone and limestone aggregate in this respect.

The high relative density results obtained in the laboratory for the slate aggregate particles correlate with the low water absorption values determined, indicating that the particles, when placed in the pavement construction, are unlikely to be significantly affected by changes in moisture conditions.

The magnesium sulphate soundness test results are comparable to conventional aggregates. This may be explained by the very low water absorption values exhibited by the aggregate therefore reducing the ingress of sulphate solution into the aggregate pores. This property of low water absorption is also thought to be the reason for the high slake durability values achieved. As the aggregates have already been fractured along the cleavage planes in the parent rock during production the property of a weak cleavage found in the parent material is not transferred to the aggregates produced.

### 9.1.2 MECHANICAL QUALITY TESTS

As discussed by Dawson (50) the strength tests carried out in the laboratory (see Section 6.3) submit aggregate particles to higher than normal stresses in an attempt to distinguish between good and bad aggregates in a short period of time. The rationale behind this approach is that the degradation is an equal and linear stress-dependent effect for all aggregates. Evidence to support this theory is not available and in the area of bulk mechanical properties where stress dependency is more understood it is non-linear and differs from aggregate to aggregate.

In a further paper by Dawson (51) it is pointed out that laboratory strength test values may have little or no relationship to the ability of an aggregate to perform under compaction. The poor relationship may be explained by the following reasons:-

i) Laboratory strength tests are carried out on a single sized fraction.

ii) These particles may be weaker than smaller particles for which weaknesses have already been exploited in their formation in the crushing process (e.g. weak cleavage).

iii) Laboratory tests submit aggregate particles to considerably higher than normal stresses.

iv) The mix of large and small particles in an unbound aggregate may be self supporting thereby avoiding problems caused by a few localized highly stressed contact points.

Thus the fact that the strength test values determined were lower than would be expected for conventional aggregates such as limestones, basalts, etc., (see Table 6.3) does not necessarily indicate poor slate aggregate performance

All the slate aggregates tested are non-plastic and have very low sulphate contents. They therefore comply with the SHW (17) requirements, and should cause no problems in these areas.

## 9.2 SLATE GRANULAR MATERIAL TEST RESULTS

The desirable properties for unbound layers in pavement construction were discussed in Section 3 and can be summarised as follows:

(a) Good load spreading ability - to protect the sub-grade

(b) Good permeability - to transfer excess water to the drainage media, thus protecting not only the sub-grade but the aggregate layer itself.

(c) Well-graded material such that small deviations in moisture content away from its optimum do not adversely affect the strength of the layer.

(d) Non-susceptibility to the action of frost if the layer concerned is within 450 mm of the surface.

(e) Resistance to permanent deformation (rutting)

One key factor for all aggregates in achieving the above properties of unbound layers is grading. Grading markedly affects stability, drainage and frost susceptibility. Thus control of grading to yield the properties sought, whether it be dense graded for maximum stability or open-graded for maximum drainage and protection from frost action, is of particular importance.

At present permeability is not specified in the SHW (17) in any layer of the pavement even though this is one of the most important functions of a granular material. Recent Department of Transport contracts (52) have specified a limit of not less than $1 \times 10^{-5}$ m/sec for granular sub-base Type 1. All the results on the range of gradings tested complied with this limit. As expected the finer gradings were less permeable than the coarser gradings.

The saturation of unbound granular materials can cause a substantial reduction in material strength and stiffness, depending on the composition of the material. Granular materials with a high coefficient of permeability will not hold moisture easily. This material would allow easier access of surface water draining under gravity to approach the water table, there would be less water in the material and the water in the capillary pores would be discontinuous.

When evaluating the coefficient of permeability from compacted samples in the laboratory it is important to attempt to correlate the results to conditions likely to be met in the field. One disadvantage of laboratory work is that the larger size fractions are usually separated out and discarded; in the present series of tests all material was passing the 20 mm sieve, whereas in highway engineering a range of particle sizes up to 75mm are used.

The main factors which affect the permeability of a granular material could be largely overcome if large samples were tested in a permeameter capable of accommodating a large range of particle sizes. The coefficients of permeability obtained from such experiments would tend to give closer results to actual field conditions.

A recent development which provides an answer to the drawbacks of the constant head method is the horizontal permeameter developed by Jones and Jones (53) and recommended in the Department of Transport Departmental Advice Note HA 41/90 (54). It has the additional advantage that it specifically measures horizontal permeability which is likely to be predominant both because of pavement drainage geometry and because of in-situ orientation of the slate particles.

The moisture density curve, including maximum dry density and optimum moisture content is influenced by a large number of variables, including material type, nature of the compaction, level of compactive effort, and the manner in which the granular material is handled before and during the compaction process. This greatly complicates the characterization of the compaction results to field situations.

The moisture-density relationship values determined on the slate sub-base aggregate are typical of granular materials in that the optimum moisture content increases with fineness of grading. Maximum dry density is achieved at a grading which corresponds to the mid-point of the specification grading for Type 1 Sub-base.

Compaction is usually achieved in the field with heavy plant whose mode of compactive effort is altered to suit the field conditions, and the material being compacted. In order to simulate field conditions in the laboratory, vibratory compaction is currently employed. One main source of error in compaction of granular materials above optimum is the loss of free water from the base of the mould. When this occurred attempts were made to collect the free water and modify the sample moisture content to allow for this, but several points were necessary to complete the curve.

Relative density has been recognised as a fundamental and practical concept for evaluating the limiting permissible drop in field density below the ultimate maximum value. In the body of an embankment which is under considerable overburden pressure, a figure of 70% relative density may be adequate, but just below the pavement a figure of 90% relative density is required to support the increased stresses being passed through the pavement from present day road traffic.

It can be seen from the frost heave test results (see Section 7.5 and Appendix) that the material has degraded during the test. This is typically the case with all aggregates and the slate appears to have suffered less degradation than many conventional aggregates. This aspect is considered to be due to the low water absorption of the slate aggregate. From the results obtained in the laboratory it can be concluded that 6% represents the maximum percentage passing the 75 micron sieve that can be tolerated if the material is to be considered as non-frost susceptible (non-proven category) in accordance with the Transport and Road Research Laboratory Report No. 829 (45).

The CBR test is entirely empirical and cannot be considered as even attempting to measure any basic property of the material. Hence the CBR of a material can only be considered as an index of its strength which, for any particular material, is dependent on the condition of the material at the time of testing.

The SHW (17) specifies a minimum CBR value of 30% for Type 2 sub-base material. The laboratory CBR values determined are greater than 30% and are typical of a good quality granular material. There is no value quoted in SHW (17) for Type 1 material.

## 9.3 COMPACTION OF SLATE GRANULAR MATERIAL

It can be concluded from the compaction trials detailed in Section 8.4 that for slate sub-base vibratory rollers are superior to smooth wheeled or grid rollers in attaining high densities and in compacting to relatively greater depths.

The purpose of compaction is to provide sub-base with stability and to increase its elastic stiffness and resistance to permanent deformation. For granular materials resistance to rutting under imposed wheel loads is of primary importance. Such resistance can be achieved with high density which in turn depends on good compaction of properly graded materials.

The material must not only be compacted to a required degree of stiffness but this deformation resistance must be maintained at an acceptable level during the life of the pavement. Thus weakening by saturation with water during wet periods or by capillary and frost action must be prevented. The high permeability of slate sub-base is therefore a beneficial factor in achieving the required deformation resistance.

## 9.4 DEGRADATION OF SLATE GRANULAR MATERIAL

The degradation process of a granular material may be considered as:-

1. The breakage of particles into approximately equal parts.
2. The breakage or crushing of angular projections which may exist on the exterior surfaces of the particles.

3. The grinding of small-scale asperities from major faces or planes of the particles (49).

The distinction between these processes is mainly in the different ratios of size of the broken product to the size of the original particle. Their relative importance differs with shape and petrology of the aggregate. For example, non-crushed rounded gravels will only experience type 1 degradation. Angular cubical particles will often experience type 2 degradation before any type 1 breakage occurs.

The initial effect of degradation of a granular material, in service is therefore likely to be a reduction in the two principal original components of stability namely rock particle interlock and interparticle surface friction between the original particles. However densifications due to the smaller fragments produced filling the spaces within the aggregate structure is likely to occur, thus producing additional points of interparticle contact within the structure at which surface friction may be mobilised. In summary the physical significance of disintegration of an aggregate structure is dependent upon the relative importance of these two changes.

It was expected that the high flakiness and elongation properties of the slate aggregate would lead to excessive breakdown of the particles after compaction. Although a visual inspection of the particles after compaction indicated that the elongated particles were fewer the degree of degradation resulting from seven years of traffic and from the monitored trials as detailed in Section 8.3 was very low and as anticipated was more evident at the coarser end of the grading (see Table 8.2). After compaction and trafficking the grading analysis of the material was still within specification limits and the small degree of breakdown did not appear to inhibit its performance as an unbound aggregate. It is generally accepted that in unbound aggregates some crushing of the aggregate under the roller is desirable because of the greater density and further frictional contacts which result. It is pointed out however that in order to be beneficial this process must take place under the roller, i.e. during initial compaction.

The greater degradation of flaky and of elongated particles of a given size often experienced is associated with the greater bending moments that can be applied to given cross sectional areas. This effect of size is a demonstration of the theory of statistical distribution of flaws, which states that the probability of a critical flaw occurring increases as the size of the member increases. It applies to irregular rock particles in a similar way as it applies to concrete cubes and beams, metal test specimens, etc. The surprisingly small degree of degradation of the slate aggregate may be explained by the high tensile strength of slate (see Section 2.2) relative to conventional aggregates allowing the slate particles to withstand relatively greater bending moments. This property of relatively high tensile strength is of course demonstrated in the use of slate as a roofing material.

Post rolling degradation under traffic inevitably leads to a certain degree of rutting due to the further compaction which takes place, independently of whether or not the final product has a higher or lower shearing resistance than the original.

The grading analyses results performed on the uncovered original slate sub-base after planing off the upper layers and the grading analyses results from the site trafficking trials indicated that minimal degradation had taken place under traffic. It should be emphasised however that the samples taken after uncovering may not be in exactly the same location as the original gradings.

The small amount of degradation that took place indicates that the current strength tests in use (Aggregate Crushing Value, Aggregate

Impact Value, and Ten per Cent Fines Value) are not necessarily able to predict how an aggregate will behave in service in an unbound granular material. Where aggregate of low crushing strength is considered for use it should be borne in mind that the amount of degradation to be expected is a function not only of the intrinsic strength for a given size and shape, but also of the aggregate grading. Clearly the standard Aggregate Crushing Value and Aggregate Impact Value tests cannot assess this factor since they are carried out on "single size" materials. A more realistic approach would be to determine properties of the unbound granular material in situ. This approach is further discussed in Section 9.5.

## 9.5 IN-SITU TESTING

A first approach to checking whether the unbound slate aggregate meets the specification and design assumptions is to determine in-situ density, moisture content, grading, and layer thickness. Of these four parameters, in practice only the in-situ dry density (and with it the moisture content) is checked. In-situ grading is at best checked on site only by looking for segregated patches. Determination of in-situ layer thickness is hardly feasible since it involves too large an inaccuracy to be of any use.

Even if all four parameters (density, moisture content, grading and layer thickness) were measured in-situ, still this first approach to checking design assumptions in-situ would be an indirect one. Density and moisture content, for instance, are by themselves of limited interest only, since they are not straightforwardly correlated to the design parameters which is of real interest, being the elastic stiffness and the resistance to permanent deformation. Furthermore, determination of density and moisture content in-situ is too cumbersome and time-consuming to allow for a sufficient number of measurements to be performed and the measurements by themselves may involve too large inaccuracies to be of practical use in acceptance control of granular layers.

A far better approach to checking the contractor's work in-situ is the direct approach of determining the elastic stiffness and the resistance to permanent deformation of the laid and compacted granular layers. The Plate Bearing Test is used in a number of countries for this purpose. This test is fundamentally sound since it involves application of realistic stresses and measurement of the resulting deformations, but it is also very cumbersome and time consuming. The speed of loading employed in the test is also incorrect but this may be relatively unimportant for dry aggregates. A clear need therefore exists to consider the use of testing procedures that combine the fundamental approach of the Plate Bearing Test with ease and speed of execution.

The following test methods allow for the checking of design assumptions with respect to the stiffness of the granular layers by means of tests that are fast and relatively simple to perform.

## FALLING WEIGHT DEFLECTOMETER (FWD)

The FWD (55) is capable of generating load pulses similar in magnitude and duration to those induced by commercial vehicles. It has the advantage that it can measure the elastic stiffness of a layer directly and approximately 100 points can be measured in one day. The equipment is relatively expensive and requires specialist operators. Difficulties are also often experienced in bedding geophone sensors onto an unbound pavement surface. Due to its high cost it would be unrealistic to expect the FWD test to be specified on a routine basis but could be used in cases of dispute.

## CLEGG IMPACT HAMMER

The main advantage of the Clegg Impact Hammer is its portability and the simplicity and speed of its use. Impact values can be specified which give a rough guide to the density and stiffness of the layer. More work needs to be done on its applicability as a quick means of proof testing of granular layers but the specification of a required "Clegg Impact Value" is an improvement over that which is used at present.

## DYNAMIC PLATE BEARING TEST

The Dynamic Plate Bearing Test (56) is a modification of the static test. The load is applied by impact loading which reduces the time required for the test and eliminates the need for a large reaction load. Although the equipment appears to be useful for design or reference purposes, it is doubtful if it could be used for routine compliance testing.

## 9.6 THE FUTURE

In 1960 a Building Research Station report (12) concluded that the limitations to slate waste utilization stemmed from economic and geographical factors. However, rapidly rising prices, resource problems, and sometimes environmental and even political pressures have reached the point where the validity of these constraints should be investigated. The interactions between the various pressures and constraints is extremely complex and the development of successful policies for more effective utilization is likely to depend on the collaboration of industry and government.

There is also a more fundamental problem which seems to call for government involvement; the long-term and sometimes nebulous benefits of utilization are not necessarily reflected in the direct and relatively short-term profitability normally required of a commercial operation. A specific proposal may not appear to be commercially attractive when viewed in terms of the usual financial criteria, although there may be considerable overall benefit to the community as a whole.

This implies that, in the United Kingdom and probably some other countries, there is a case for investigating ways in which increased public interest could stimulate the commercial sector of industry. It is not suggested that public money must be spent to provide plant or to develop transport facilities although, in some cases, there may be justification for this kind of support where the benefits are obvious and readily quantifiable. A generally more satisfactory approach is to create a more responsive environment. To take an example, the dual tendering system, used for new road contracts in the United Kingdom, involves the assessment of using wastes as alternatives to other materials for bulk fill (57).

In order to develop new uses or extend existing ones, it is necessary to overcome the effects of the inhibitory factors discussed in Section 9.5. Engineers will only use materials if they have sufficient confidence in them; and committees which are responsible for drafting standards will only revise them if there is sufficient information to show that this can be done safely. This points to the need for making use of in situ testing as discussed in Section 9.4 in order to determine the elastic stiffness and permanent deformation of the laid and compacted granular layers, allowing the engineer to check that the specification and design is being adhered to, but, at the same time, ensuring that any material can be used providing it performs adequately.

# 10 CONCLUSIONS

1. The construction industry is consuming increasing quantities of raw materials at the same time as seeking to conserve its natural resources. The use of slate waste as an unbound granular material can assist to resolve this paradox.

2. It is in the national interest to make use of waste materials as alternatives to conventional aggregates as this conserves the supplies of naturally occurring aggregates and assists in problems arising from the disposal of unwanted materials.

3. There is a case for investigating ways in which increased public interest with regard to the utilization of slate waste could stimulate the commercial sector of industry by creating a more responsive environment.

4. Careful selecting and crushing of slate waste produces a material which complies with the requirements of Granular Sub-Base Type 1 and Type 2, in accordance with the specification for Highway Works (17).

5. Slate waste is used as an unbound granular material in the United States, and is accepted as satisfactory.

6. Slate granular material has been used successfully on the Bangor By-Pass and other road contracts in North Wales.

7. The mechanical and physical properties of the slate aggregates from different sources are reasonably consistent despite the variation in geology.

8. The strength tests carried out in the laboratory indicate that slate aggregate has a lower strength than conventional aggregates. This does not appear to inhibit its performance as an unbound granular material.

9. Slate aggregates have a considerably higher flakiness index than conventional aggregates, however site trials have indicated that this property does not lead to increased degradation or instability during the compaction process.

10. Water absorption values of slate aggregates are lower than conventional aggregates indicating good durability.

11. Permeability test results on a range of gradings covering the Granular Sub Base Type 1 specifications (17) indicate compliance with recent Department of Transport permeability specifications. However tests have not been carried out with the horizontal permeameters (49).

12. Frost Susceptibility test results on a range of gradings indicated that 6% represents the maximum percentage passing the 75 micron sieve that can be tolerated if the material is to be considered non frost susceptible (non-proven category) in accordance with the Transport and Road Research Laboratory Report No. 829.(42)

13. Due to the better control now available over selecting and crushing of slate waste to produce an unbound granular material, it is no longer necessary to use a combination of grid and vibratory rollers for compaction. In situ tests indicate that the material can be adequately compacted by vibratory rollers alone.

14. Due to the stigma associated with the word "waste" it is recommended that in-situ testing is carried out on the unbound granular material to check that the design parameters are being

complied with. This implies the specification of end product testing as opposed to a method specification.

15. Apart from some exceptions, there seems to be little evidence that the utilization of slate waste as an unbound granular material is reaching its potential.

16. The distance of slate waste sources from potential markets inhibits its use.

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Quarrying
Processing
Slate (General)
Waste Utilisation
Slate Statistics
United Kingdom Slate Geology

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# APPENDIX A

## TABLE A.1. DETAILED TEST RESULTS

## (FLAKINESS AND ELONGATION INDEX)

| SOURCE | PENRHYN | | FFESTINIOG | | LLECHWEDD | | CROES-YR-DDWY-AFON | | ABERLLEFENI | | BURLINGTON | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | F | E | F | E | F | E | F | E | F | E | F | E |
| SIZE FRACTION (mm) | | | | | | | | | | | | |
| 63.0-50.0 | 96 | - | 100 | - | 100 | - | 100 | - | 93 | - | 96 | - |
| 50.0-37.5 | 94 | 26 | 100 | 33 | 100 | 36 | 100 | 31 | 96 | 24 | 99 | 24 |
| 37.5-28.0 | 95 | 21 | 100 | 37 | 100 | 37 | 100 | 33 | 94 | 23 | 97 | 26 |
| 28.0-20.0 | 90 | 25 | 100 | 24 | 100 | 35 | 100 | 35 | 91 | 21 | 100 | 27 |
| 20.0-14.0 | 91 | 27 | 100 | 28 | 100 | 31 | 100 | 37 | 95 | 21 | 98 | 28 |
| 14.0-10.0 | 98 | 20 | 100 | 27 | 100 | 34 | 100 | 38 | 90 | 26 | 100 | 29 |
| 10.0-6.3 | 95 | 21 | 100 | 27 | 100 | 33 | 100 | 31 | 92 | 24 | 96 | 25 |

F = Flakiness Index

E = Elongation Index

## APPENDIX B

## GRADATION COEFFICIENTS

Hazens effective size $= D_{10}$ is the maximum particle size of the smallest ten per cent

Coefficient of Uniformity $C_U = \dfrac{D_{60}}{D_{10}}$ is the ratio of the maximum particle size of the smallest sixty per cent to the effective size.

Coefficient of curvature $C_C = \dfrac{D_{30}^{2}}{D_{60} \times D_{10}}$

is the ratio of the maximum particle size of the smallest 30 per cent squared to

the production of $D_{60}$ by $D_{10}$.

# APPENDIX C

## TABLE C.1 BREAKDOWN OF MATERIAL AFTER FROST TEST

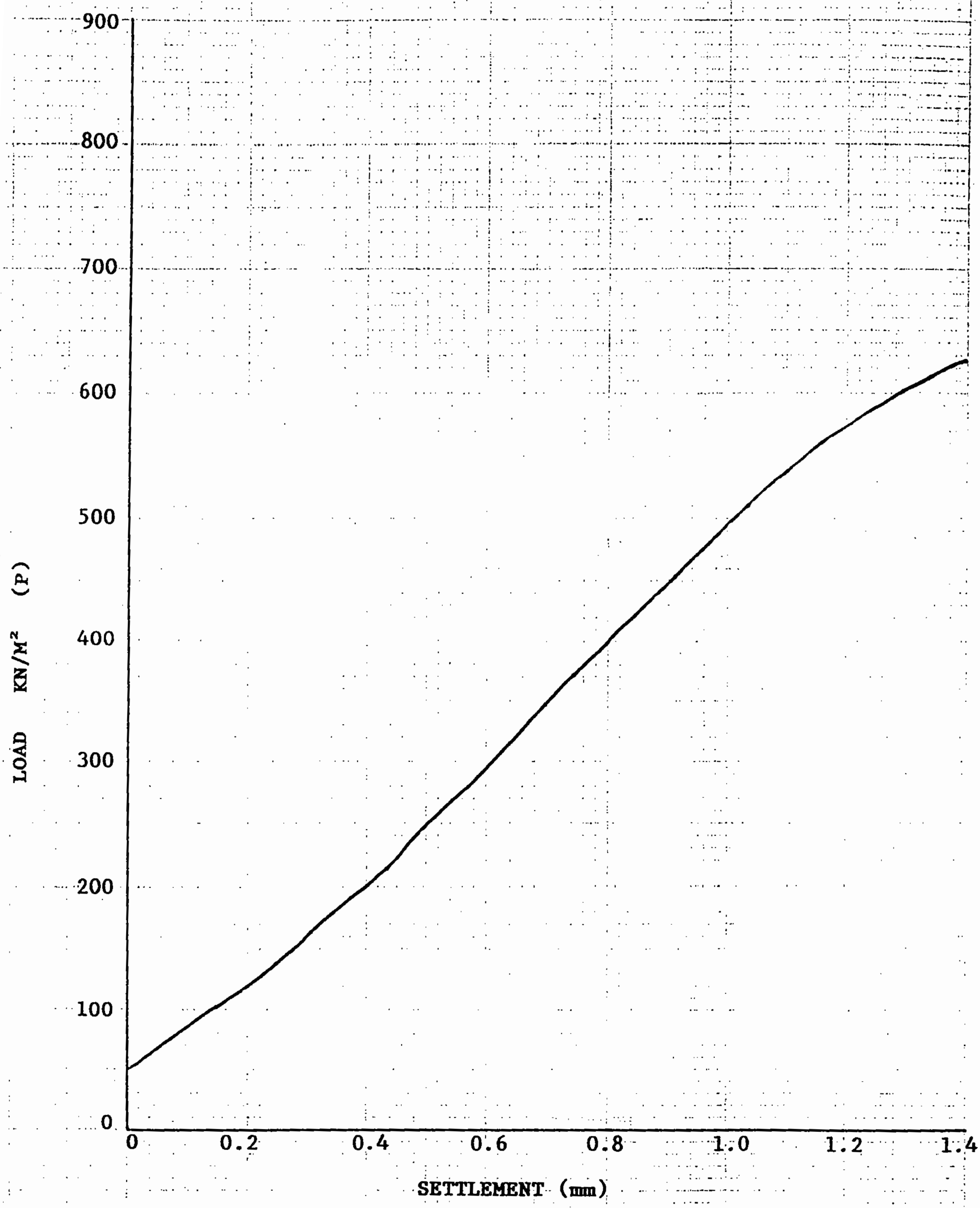
| SAMPLE NO. | A | B | C | D | E | F | G | H |
|---|---|---|---|---|---|---|---|---|
| BS SIEVE SIZE (mm) | PERCENTAGE PASSING (TEST GRADING) | | | | | | | |
| 75.0 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 37.5 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 20.0 | 64 | 67 | 70 | 73 | 76 | 69 | 84 | 87 |
| 10.0 | 40 | 44 | 48 | 52 | 56 | 60 | 65 | 70 |
| 5.0 | 25 | 27 | 29 | 31 | 34 | 37 | 41 | 45 |
| .600 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 |
| .75 | 3.2 | 4.3 | 5.1 | 6.3 | 7.1 | 8.2 | 9.1 | 10.0 |
| BS SIEVE SIZE (mm) | PERCENTAGE PASSING (AFTER FROST TEST) | | | | | | | |
| 75.0 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 37.5 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 20.0 | 64 | 67 | 74 | 78 | 77 | 80 | 86 | 87 |
| 10.0 | 44 | 48 | 51 | 54 | 59 | 65 | 70 | 74 |
| 5.0 | 30 | 32 | 35 | 37 | 39 | 44 | 46 | 51 |
| .600 | 12 | 15 | 18 | 19 | 21 | 24 | 25 | 26 |
| .75 | 6.9 | 7.4 | 7.9 | 9.3 | 11.3 | 12.0 | 12.4 | 13.6 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| TEST DENSITY (kg/m$^{3}$) | 2210 | 2230 | 2250 | 2250 | 2240 | 2200 | 2200 | 2200 |
| TEST MOISTURE CONTENT (kg/m$^{3}$) | 3.8 | 3.8 | 3.9 | 4.1 | 4.3 | 4.5 | 4.5 | 4.6 |
| FROST HEAVE (mm) | 3 | 4 | 4 | 6 | 8 | 12 | 17 | 22 |

# APPENDIX D

## PLATE BEARING TEST RESULTS

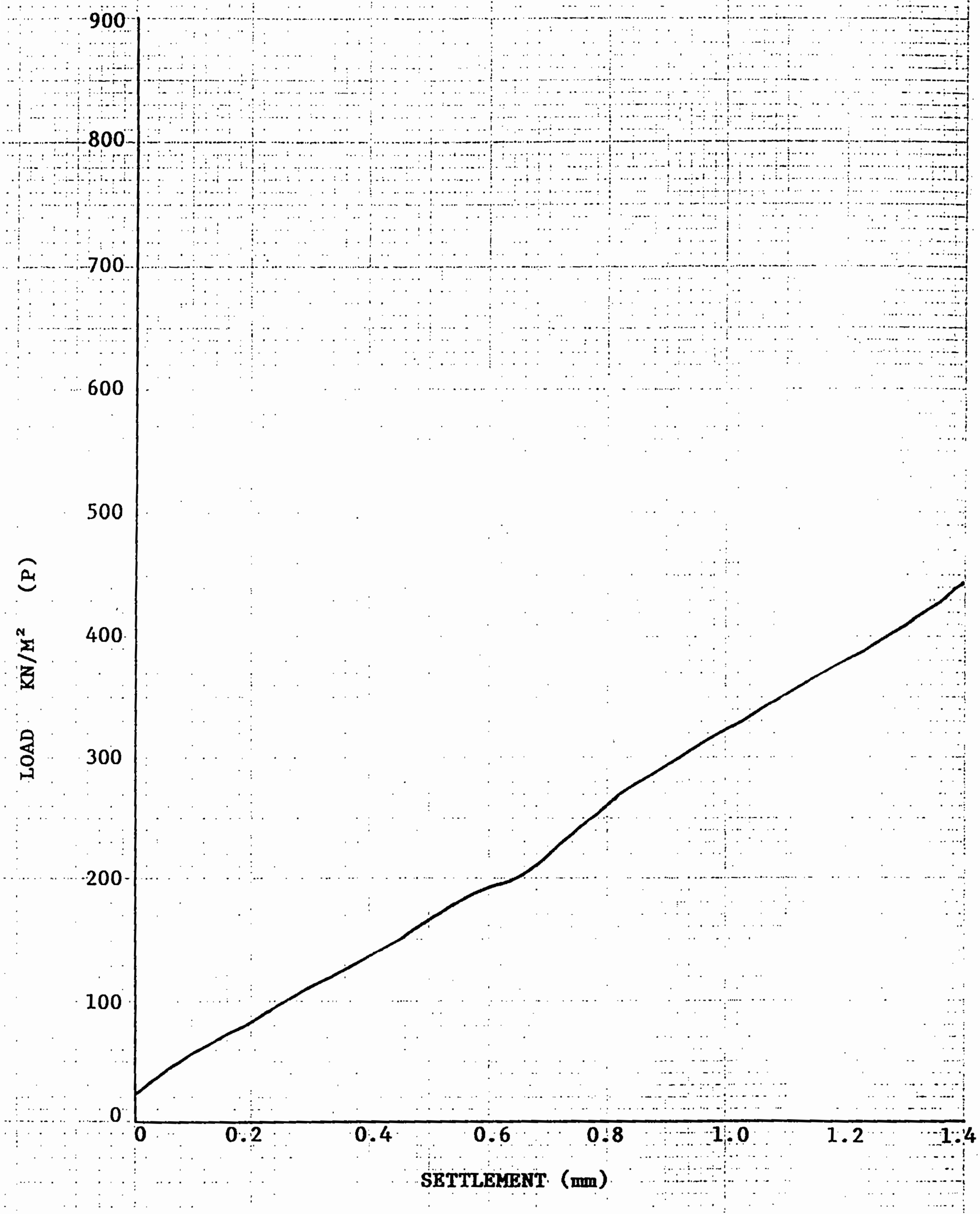
All the following tests were carried out on the Bangor By-Pass




LOCATION: A

MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 461 kNmm/$M^2$

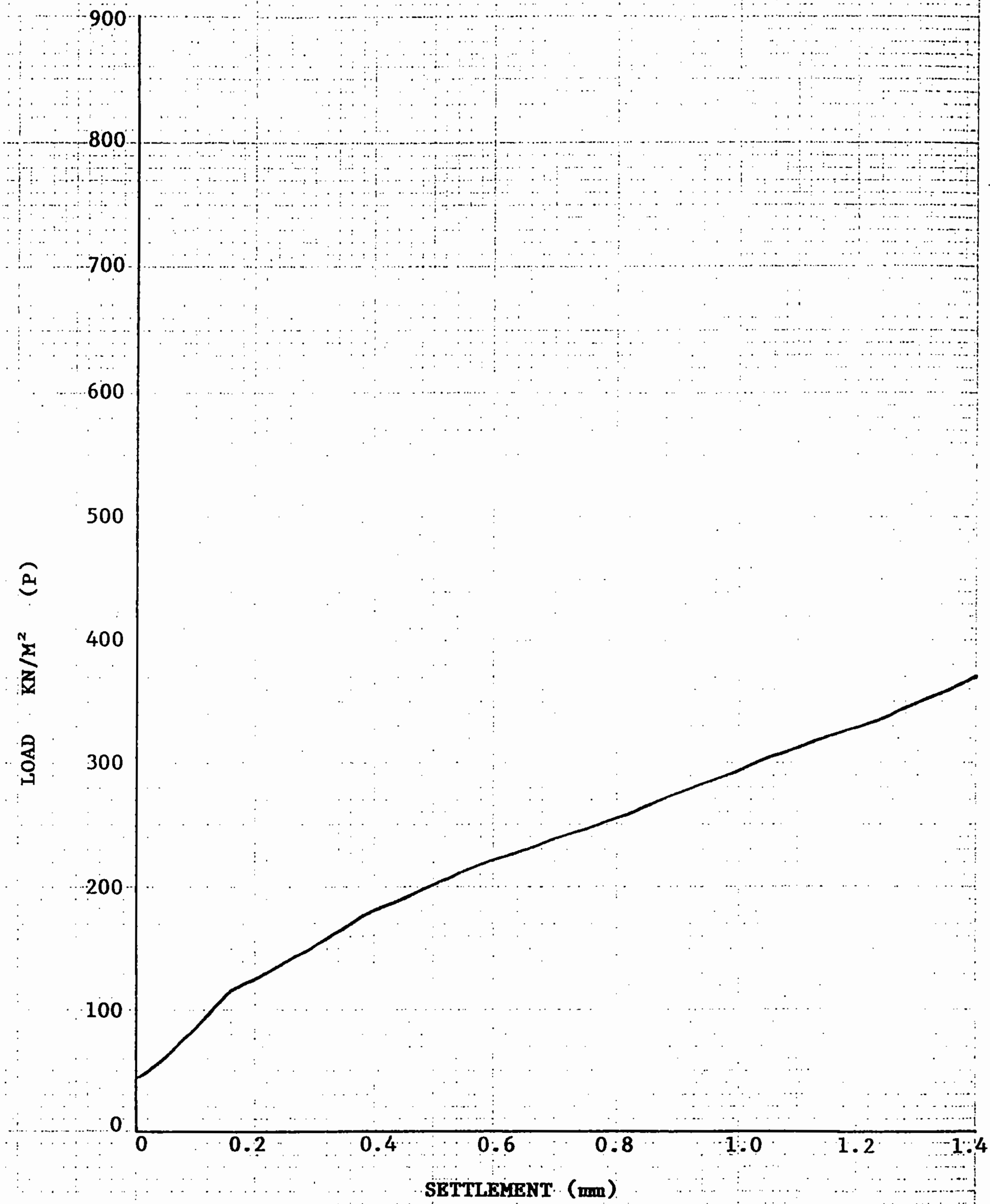
EQUIVALENT C.B.R = 42




LOCATION: B

MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 315 kNmm/M²

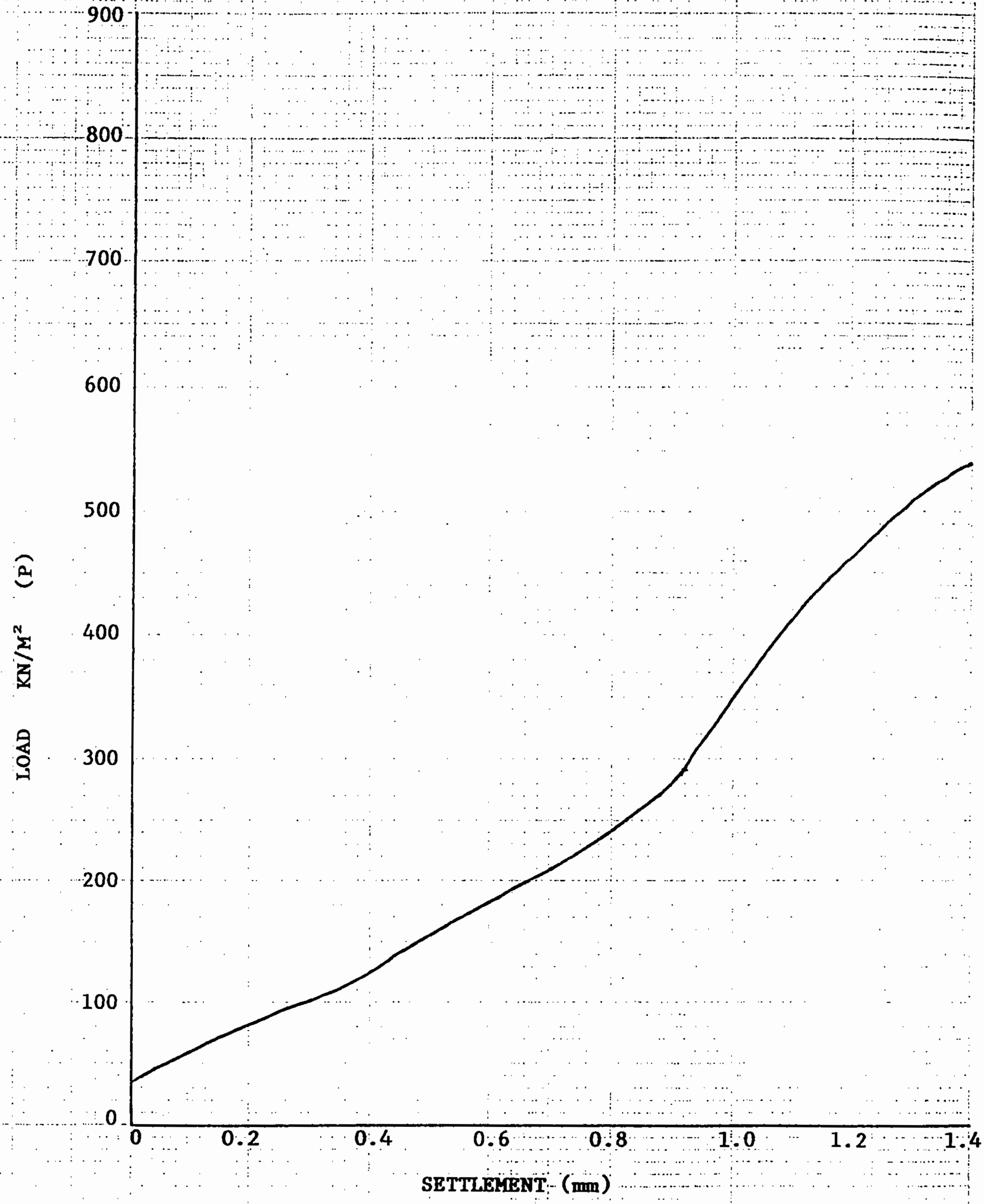
EQUIVALENT C.B.R = 26




LOCATION: C

MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 269 kNmm/M²

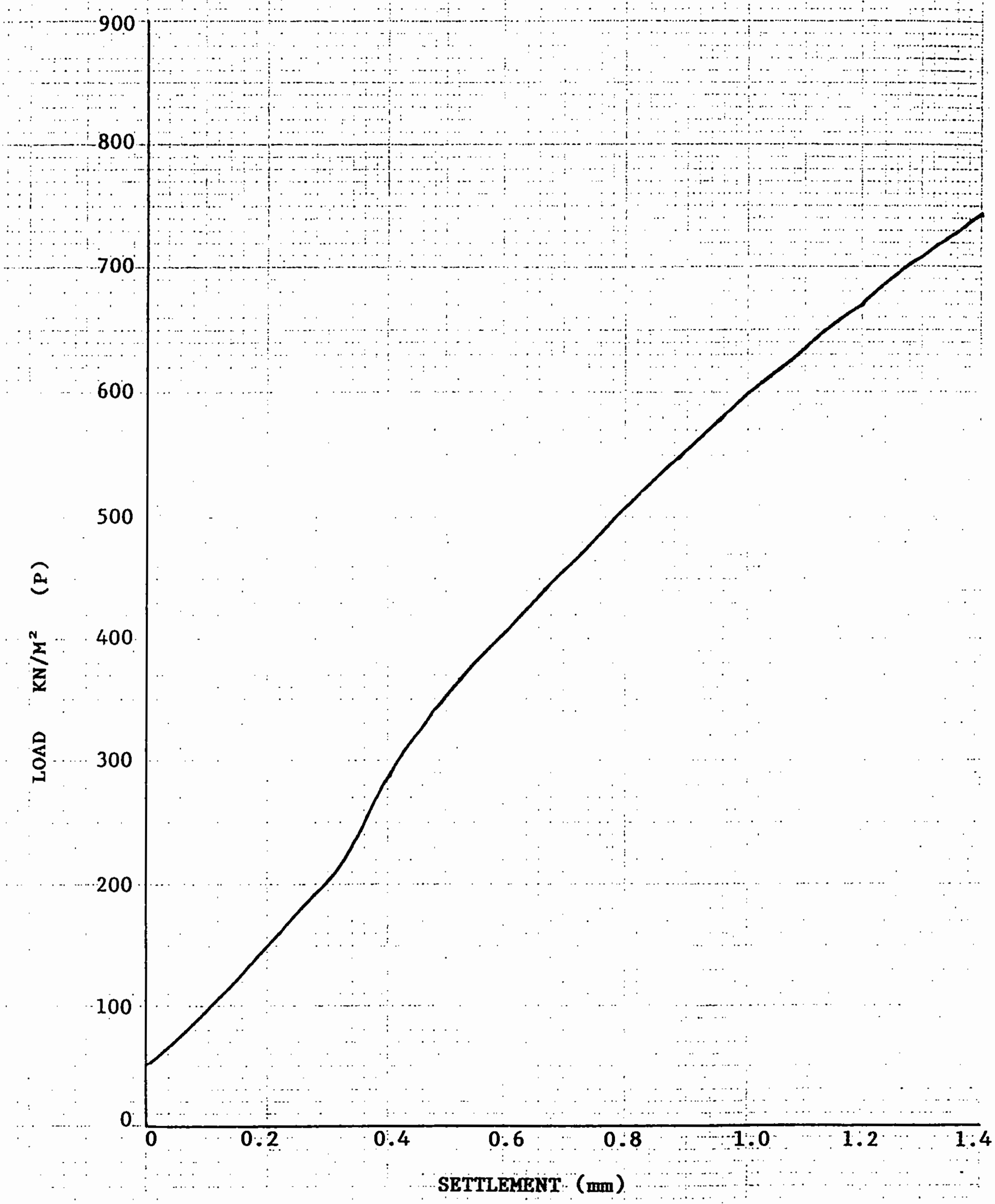
EQUIVALENT C.B.R = 23




LOCATION: D

MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 385 kNmm/M²

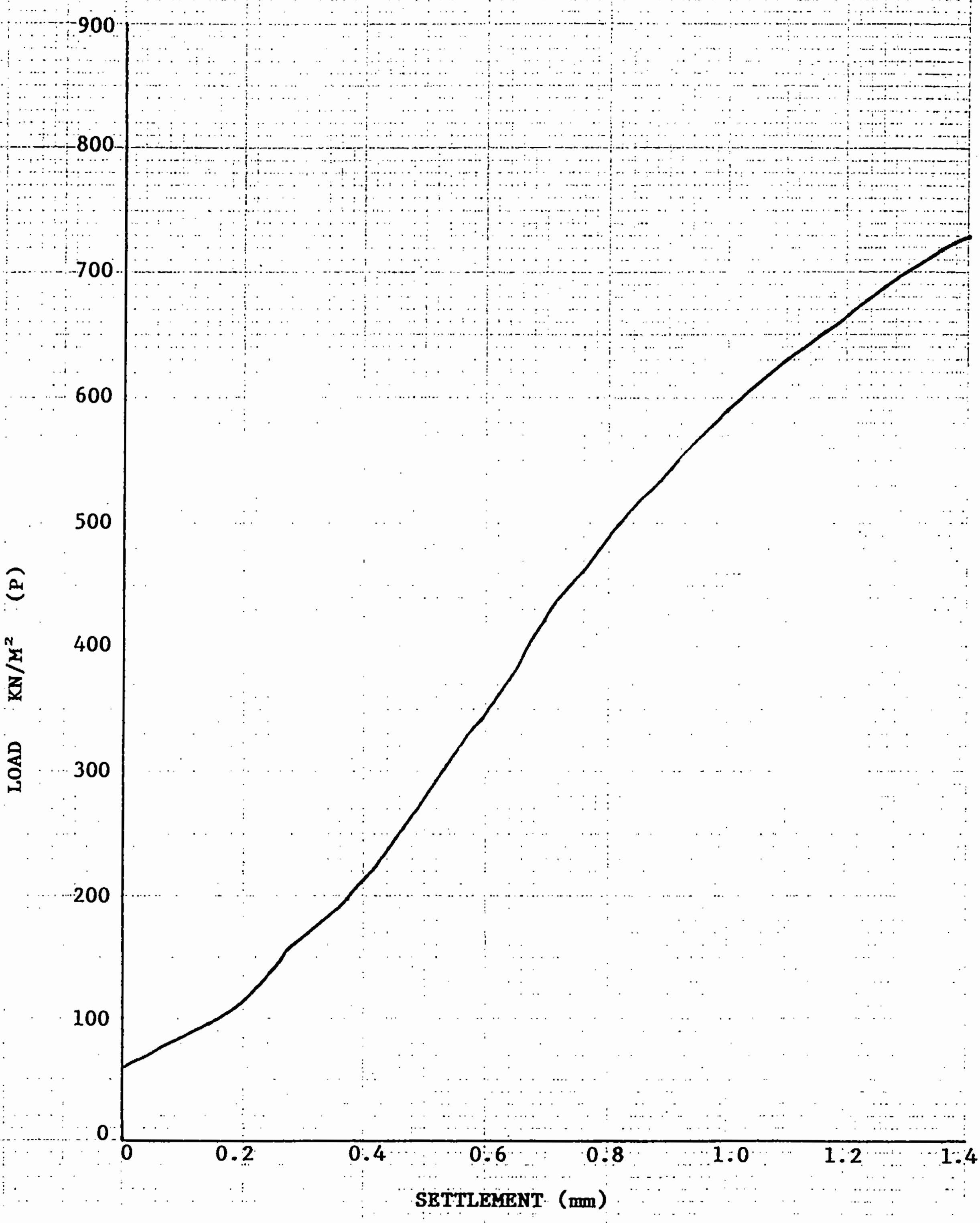
EQUIVALENT C.B.R = 34




LOCATION: F

MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 546 kNmm/$M^2$

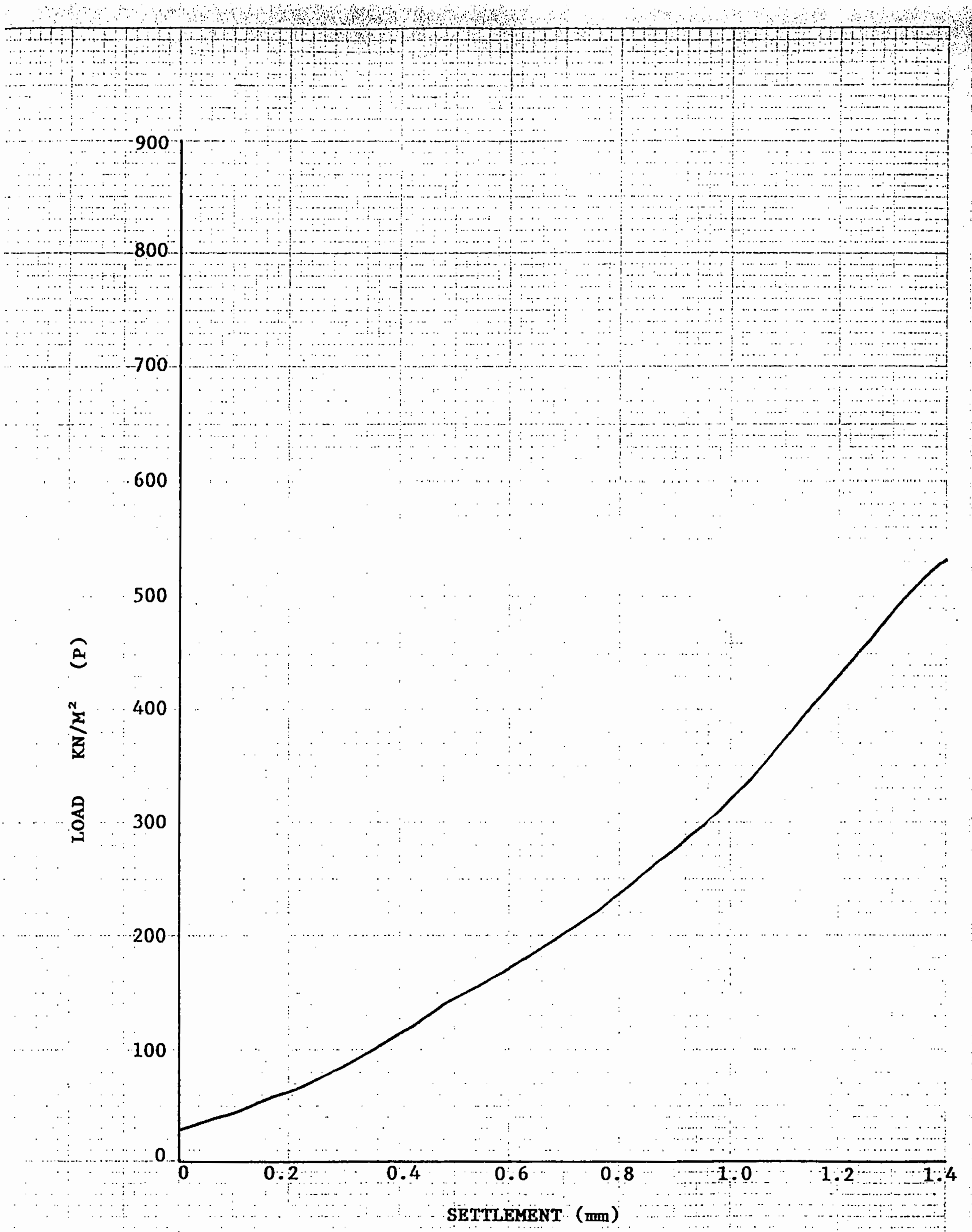
EQUIVALENT C.B.R = 57




LOCATION: G

MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 538 kNmm/$M^2$

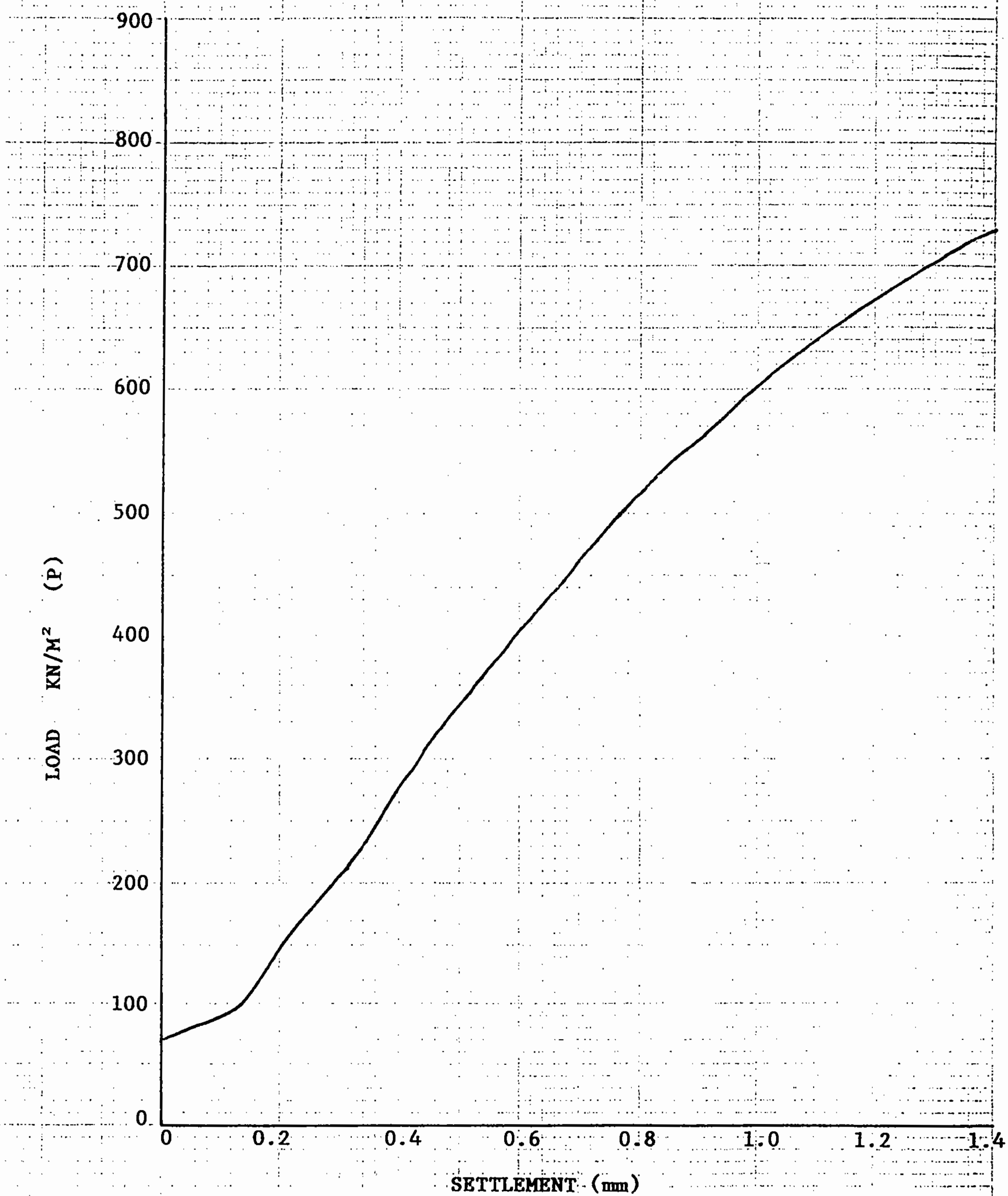
EQUIVALENT C.B.R = 55




**LOCATION: H**

**MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 378 kNmm/M²**

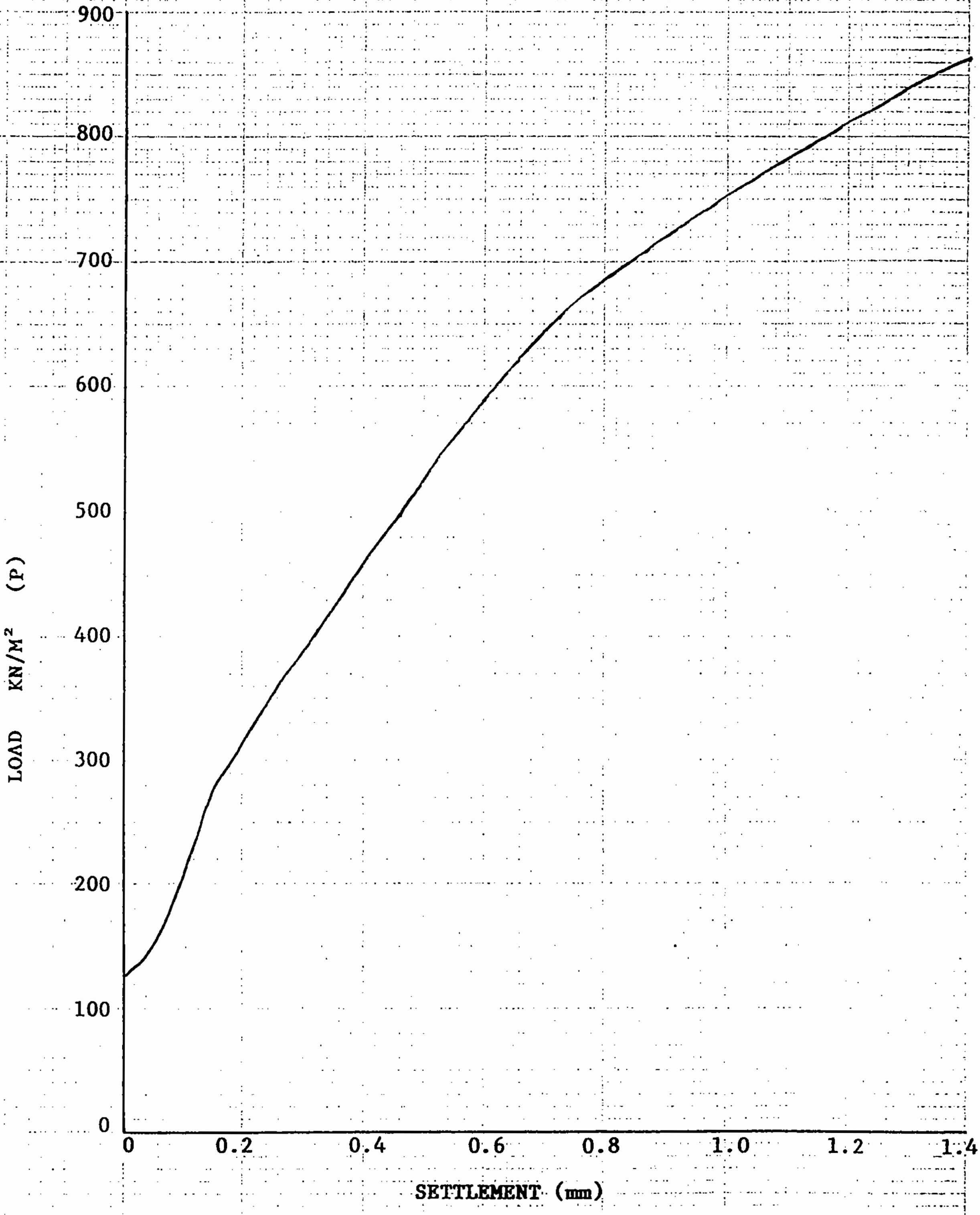
**EQUIVALENT C.B.R = 34**




LOCATION: I

MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 538 kNmm/M²

EQUIVALENT C.B.R = 55




LOCATION: J

MODULUS OF SUB-GRADE REACTION (k) = p/1.3 = 646 kNmm/M²

EQUIVALENT C.B.R = 72

# APPENDIX E

TABLE E.1 Grading analysis of original Type 1 sub-base material after approximately seven years of traffic

| Sample.No | A | B | C | D | E | F | G | H | I | J |
|---|---|---|---|---|---|---|---|---|---|---|
| BS Sieve Size | % Passing (Original Gradings) | | | | | | | | | |
| 75.0 mm | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 37.5 mm | 100 | 97 | 100 | 92 | 96 | 94 | 100 | 100 | 98 | 100 |
| 20.0 mm | 86 | 84 | 74 | 62 | 77 | 71 | 80 | 72 | 69 | 74 |
| 10.0 mm | 61 | 60 | 58 | 47 | 52 | 50 | 58 | 60 | 49 | 50 |
| 5.0 mm | 29 | 30 | 31 | 28 | 32 | 33 | 35 | 32 | 27 | 33 |
| 600 micron | 9 | 10 | 9 | 11 | 12 | 11 | 10 | 9 | 9 | 9 |
| 75 micron | 3.2 | 3.7 | 3.6 | 4.1 | 4.0 | 3.9 | 4.7 | 3.6 | 4.1 | 4.3 |
| | % Passing (Uncovered Gradings) | | | | | | | | | |
| 75.0 mm | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 37.5 mm | 98 | 100 | 97 | 96 | 97 | 100 | 100 | 100 | 100 | 100 |
| 20.0 mm | 82 | 87 | 84 | 79 | 74 | 86 | 81 | 77 | 73 | 66 |
| 10.0 mm | 67 | 63 | 67 | 54 | 59 | 56 | 56 | 61 | 63 | 63 |
| 5.0 mm | 31 | 33 | 31 | 30 | 30 | 33 | 32 | 29 | 34 | 36 |
| 600 micron | 11 | 12 | 10 | 11 | 11 | 12 | 10 | 9 | 10 | 10 |
| 75 micron | 4.0 | 3.9 | 4.0 | 3.6 | 3.9 | 4.1 | 4.7 | 4.4 | 4.6 | 4.3 |

TABLE E.2 Comparison of grading analyses after 1, 2, 4, and 6 passes with a vibratory roller (Samples K, L, M and N).

a)

| SAMPLE NUMBER K | | | | | |
|---|---|---|---|---|---|
| No. of Roller Passes | 0 | 1 | 2 | 4 | 6 |
| BS Sieve Size | PERCENTAGE PASSING | | | | |
| 75.0 mm | 100 | 100 | 100 | 100 | 100 |
| 37.5 mm | 98 | 100 | 98 | 99 | 100 |
| 20.0 mm | 76 | 76 | 75 | 77 | 78 |
| 10.0 mm | 54 | 55 | 54 | 56 | 56 |
| 5.0 mm | 30 | 31 | 31 | 30 | 31 |
| 600 micron | 9 | 9 | 9 | 10 | 10 |
| 75 micron | 3.4 | 3.3 | 3.5 | 3.5 | 3.7 |

b)

| SAMPLE NUMBER L | | | | | |
|---|---|---|---|---|---|
| No. of Roller Passes | 0 | 1 | 2 | 4 | 6 |
| BS Sieve Size | PERCENTAGE PASSING | | | | |
| 75.0 mm | 100 | 100 | 100 | 100 | 100 |
| 37.5 mm | 100 | 100 | 100 | 100 | 100 |
| 20.0 mm | 80 | 81 | 86 | 83 | 83 |
| 10.0 mm | 59 | 61 | 60 | 63 | 61 |
| 5.0 mm | 34 | 34 | 33 | 36 | 35 |
| 600 micron | 10 | 10 | 10 | 11 | 11 |
| 75 micron | 3.9 | 3.9 | 3.8 | 4.0 | 4.0 |

c)

| SAMPLE NUMBER M | | | | | |
|---|---|---|---|---|---|
| No. of Roller Passes | 0 | 1 | 2 | 4 | 6 |
| BS Sieve Size | PERCENTAGE PASSING | | | | |
| 75.0 mm | 100 | 100 | 100 | 100 | 100 |
| 37.5 mm | 92 | 93 | 93 | 94 | 96 |
| 20.0 mm | 71 | 71 | 73 | 73 | 72 |
| 10.0 mm | 49 | 52 | 51 | 49 | 52 |
| 5.0 mm | 27 | 28 | 28 | 29 | 29 |
| 600 micron | 9 | 9 | 9 | 9 | 9 |
| 75 micron | 3.1 | 3.3 | 3.4 | 3.4 | 3.4 |

d)

| SAMPLE NUMBER N | | | | | |
|---|---|---|---|---|---|
| No. of Roller Passes | 0 | 1 | 2 | 4 | 6 |
| BS Sieve Size | PERCENTAGE PASSING | | | | |
| 75.0 mm | 100 | 100 | 100 | 100 | 100 |
| 37.5 mm | 91 | 93 | 95 | 95 | 96 |
| 20.0 mm | 73 | 77 | 75 | 76 | 76 |
| 10.0 mm | 50 | 50 | 49 | 48 | 49 |
| 5.0 mm | 28 | 29 | 28 | 31 | 30 |
| 600 micron | 9 | 9 | 9 | 10 | 10 |
| 75 micron | 3.0 | 3.1 | 3.3 | 3.4 | 3.4 |

# APPENDIX F

TABLE F.1 Results of in-situ dry density tests carried out during compaction

| LOCATION | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
|---|---|---|---|---|---|---|---|---|---|---|
| SAMPLE NO. | K | | L | | M | | N | | O | |
| NO OF ROLLER PASSES | IN-SITU DRY DENSITY ( $kg/m^3$ ) | | | | | | | | | |
| 1 | 1785 | 1800 | 1770 | 1750 | 1790 | 1810 | 1815 | 1795 | 1815 | 1805 |
| 2 | 1990 | 2030 | 2035 | 2015 | 2050 | 2030 | 2095 | 2100 | 2095 | 2100 |
| 4 | 2250 | 2230 | 2240 | 2245 | 2245 | 2235 | 2230 | 2240 | 2250 | 2255 |
| 6 | 2260 | 2250 | 2240 | 2245 | 2250 | 2240 | 2240 | 2240 | 2260 | 2265 |

# APPENDIX G

## SPECIFICATION FOR UNBOUND GRANULAR MATERIALS GRANULAR SUB-BASE MATERIAL TYPE 1.

Granular Sub-Base Material Type 1. is specified in Clause 803 of the specification for Highway Works (17) which requires that:-

"(a) The granular material shall be crushed rock, crushed concrete or well burnt non-plastic shale.

(b) The material shall be well graded and lie within the grading envelope shown in Table 8/20 of the SHW as reproduced below

| BS Sieve Size | Percentage by mass passing |
|---|---|
| 75 mm | 100 |
| 37.5 mm | 85 - 100 |
| 10 mm | 40 - 70 |
| 5 mm | 25 - 45 |
| 600 micron | 8 - 22 |
| 75 micron | 0 - 10 |
| N.B. The particle size shall be determined by the washing and sieving method of BS 812:Part 103. | |

Sub-base Type 1 limits of grading.

(c) The material passing the 425 micron BS sieve shall be non-plastic as defined in BS 1377 and tested in compliance therewith.

(d) The material shall be transported, laid and compacted without drying out or segregation.

(e) The material shall have a 10% fines value of 50 kN or more when tested in compliance with BS 812 in a saturated and surface dried condition. Prior to testing, the selected test portions shall be soaked in water at room temperature for 24 hours without previously having been oven dried."

GRANULAR SUB-BASE MATERIAL TYPE 2.

Granular Sub-Base Material Type 2. is specified in Clause 804 of the specification for Highway Works which requires that:-

"(a) The granular material shall be natural sands, gravels, crushed slag, crushed rock, crushed concrete or well burnt non-plastic shale.

(b) The material shall be well graded and lie within the grading envelope shown in Table 8/2 of the SHW as reproduced overleaf:-

| BS Sieve Size | Percentage by mass passing |
|---|---|
| 75 mm | 100 |
| 37.5 mm | 85 - 100 |
| 10 mm | 45 - 100 |
| 5 mm | 25 - 85 |
| 600 micron | 8 - 45 |
| 75 micron | 0 - 10 |
| N.B. The particle size shall be determined by the washing and sieving method of BS 812:Part 103 | |

Sub-base Type 2 limits of grading.

(c) The material passing the 425 micron BS sieve when tested in compliance with BS 1377 shall have a plasticity index of less than 6.

(d) The material shall satisfy the minimum CBR requirement of Appendix 7/1 of SHW for the particular contract when tested in accordance with BS 1377 Test 16 using surcharge discs. The test should be carried out at the density and moisture content likely to develop in equilibrium pavement conditions. These should be taken as being the density relating to an air voids content of 5% and optimum moisture content determined in compliance with BS 5835.

(e) The material shall be transported, laid and compacted at a moisture content within the range 1% above to 2% below optimum moisture content determined in compliance with BS 5835 and without drying out or segregation.

(f) The material shall have a 10% fines value of 50 kN or more when tested in compliance with BS 812 in a saturated and surface dried condition. Prior to testing, the selected test portions shall be soaked in water at room temperature for 24 hours without previously having been oven dried."

CAPPING MATERIALS

Capping materials are specified in Clause 613 of the specification for Highway Works which requires that :-

"(a) The granular material shall be natural sands, gravels, crushed rock, crushed slag, crushed concrete or colliery shale.

(b) The material shall comply with the grading envelope in Table 6/1 of the SHW for either Class 6F1 or 6F2 reproduced below:-

| BS Sieve Size | Percentage by mass Passing | |
|---|---|---|
| | 6F1 | 6F2 |
| 125 mm | | 100 |
| 90 mm | | 80 - 100 |
| 75 mm | 100 | 65 - 100 |
| 37.5 mm | 75 - 100 | 45 - 100 |
| 10 mm | 40 - 95 | 15 - 60 |
| 5 mm | 30 - 85 | 10 - 45 |
| 600 micron | 10 - 50 | 0 - 25 |
| 63 micron | 0 - 15 | 0 - 12 |
| N.B. The particle size shall be determined by the washing and sieving method of BS 1377 Test 7. | | |

Capping Material Limits of Grading

(c) 6F1 material shall have a 10% fines value of 30 kN or more when tested in compliance with BS 812 in a saturated and surface dried condition. Prior to testing, the selected test portions shall be soaked in water at room temperature for 24 hours without previously having been oven dried. The 10% fines value required for 6F2 material varies with contract specifications.

(e) The material shall be transported, laid and compacted at a moisture content within the range optimum moisture content to 2% below optimum moisture content determined in compliance with BS 1377 Test 14."