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PARTICLE INTERACTION AND THE STABILITY
OF ASPHALT MIXES

by

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ABSTRACT

The thesis presents details of tests carried out to assess the effect of particle interaction on the mechanical properties of dense asphalt mixes.

The degrees of particle interaction of a range of fine granular materials have been assessed quantitatively on the basis of packing characteristics and also the rate of discharge from an orifice. These have been shown to be related to the mechanical properties of sand asphalts assessed by means of the Marshall test.

Tests have also been carried out to investigate the effects of aggregate type and grading on the properties of stone filled mixes, and their influence on Marshall stability, flow and void characteristics have been discussed. In particular, the separate contributions of the stone and sand to the load bearing properties of the mix have been examined, especially in relation to the shape of the Marshall test curves over a wide range of binder contents. Particular reference has been made to the behaviour of mixes in the range of binder contents below those more generally examined.

The review of previous work includes a detailed examination of the mechanical tests available for bituminous materials, and a study of the various techniques provides a basis for assessing the mechanical properties of a material considered most relevant in its performance under load. The application of the various test procedures to mix design has been considered and the merits of the Marshall test discussed.

Recommendations are given for an extension of the work reported, with especial reference to the correlation of results of laboratory tests on bituminous materials with their behaviour in road surfaces.
The work reported in this thesis was carried out in the Highway Engineering Materials Laboratory of Battersea College of Technology, now the University of Surrey.

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CHAPTER 1

INTRODUCTION.

A. A review of developments and trends in highway pavement construction.

Historical review of methods of pavement construction.

The design, construction and maintenance of roads has exacted the attention of civil engineers since the beginning of civilization, but until recently, practice has been to improve the bearing capacity of existing constructions rather than to construct anew. The recent need for new highway systems has enabled a more scientific approach to be taken in the design of road pavements, especially in the selection of materials and constructional techniques, and this has lead to economic structures capable of withstanding the heavy demands of modern traffic.

The Romans were undoubtedly the first great road engineers, good roads being a necessity for the expansion and development of their Empire. Road engineering was a respected vocation, and the military importance of these roads required a high standard of construction and maintenance.

In the construction of their roads, the first stage was to cut two parallel trenches along the proposed edges of the formation, to excavate the material between down to hard ground, and to fill with fine earth compacted to form the "pavimentum". A layer of small squared stones was then placed, either dry, or bonded with mortar, to provide a watertight "statumen", and a lime stone "concrete" placed on this forming the "rudus". The next layer, the "muculis", was of low grade concrete, consisting of broken tile, gravel or sand, and lime and clay, surfaced with either fitted stones, or, for less
SUMMUM DORSUM
NUCLEUS

STATUMEN PAVIMENTUM RUDUS

ROMAN CAUSEWAY CONSTRUCTION

WEARING COURSE BASE COURSE

FORMATION LEVEL SUB GRADE SUB BASE BASE

MODERN HIGHWAY CONSTRUCTION
heavily trafficked roads, a gravel/lime cement. This crushed wearing surface was known as the "summun dorum". The great care taken to ensure that water did not enter the formation resulted in a durable construction; many roads have lasted for almost 200 years, some, forming the foundations of modern highways.

The Roman roads constructed in this country provided passage long after their builders had left, and for centuries remained the only engineered roads, other secondary systems developing out of local lanes and tracks.

The state of roads deteriorated to such an extent that in 1656 an Act was passed attempting to co-ordinate and organise road maintenance by delegating responsibility to locally appointed Surveyors of Roads, requiring the local inhabitants to maintain their own portions of highway. This scheme did not prove successful, and road conditions deteriorated still further until 1663 when the first Turnpike Act was passed, "For repairing the High-ways within the Counties of Hertford, Cambridge and Huntingdon". This Act established the practice of tolls providing a modest income by no means sufficient for the maintenance of the roads, but led to the beginnings of a professional approach to road maintenance and the establishment of salaried officials.

A lead in new methods of road design was taken by the French who had been working on road construction since the early 17th century, and in 1664, Tresaguet, of the "Ecole des Ponts et Chaussées", constructed the first well engineered road of modern times, depending upon an impervious surface to protect a dry bed. Tresaguet's road consisted of a cambered formation upon which was laid a foundation of stones placed on edge to a depth of some 1½ to 2 inches. Then was a layer of slightly smaller stones laid to about the same thickness, surfaced with a 3'-layer of small stones referred to as "broken to about the size of a walnut". This surface was protected
to form a hard impervious protection for the road
practised by the Romans and still recommended.

In his paper to the Royal Society in 1692, he
proposed that roads should consist of a base of
sand, a principle followed by MacAdam himself
and Wilson. James Macadam himself followed Phillips' proposals in building
his first road in Yorkshire with notable success.

In the early 19th century, Telford supervised the con-
struction of many miles of road in Scotland. The foundation,
which rested upon a base of sand, comprised a bed of stones placed in groups
of approximately 7 inches, but on a level, rather than stacked
on top of one another. The compacted stone had a density of one and a half in their largest diameter. These
stones, one inch thick, were laid in two layers, the upper layer
forming a surface of 12 inches, whereas the lower layer
made up the base. In addition, small stones were
used for the upper layer, at intervals of one hundred yards.

John MacAdam had written a number of works
that "it is the native soil which forms the
surface that should be preserved in the
road, weight without sinking". New
methods were developed to ensure a dry foundation
of 7-inch stone, each layer
laid
coarser. This construction was then
levelled in the road surface, chalk
but rolled upon the iron road
rock to fill up the voids, itself.

Although none continue construction.
to be more durable than modern asphalt on present day roads.

A patent for the extraction of asphaltum was taken out in 1861, but it was not until the present
the beginning of the 19th century that the construction. Tar was used mainly
impregnation of tar-bound surfaces and as
preparation of roads in its early
the asphaltic concrete was subsequently
and on roads introduced in 1892, but tar was used
asphaltic concrete for pavements and roads was

The development of asphalt in the early 19th century was
possibility of asphalt. The first use of asphalt in Austria was in 1765, and subsequently in several
Evidence of the use of asphalt was discovered in the 18th century as a result of the...
where paving slabs were jointed with a pitch. Numerous processional routes in Babylon were 1 to 2½ yards wide and were filled in a similar manner in about 2000 B.C. The pitch was found over extensive deposits in the Near East, occurring in bitumen, asphalt, pitch, pitchy rocks or asphaltites, and coal. However, most of these early civilizations used the pitch alone for their pavements, not reaped till the 1700s for generating energy, when introduced into London in 1700, but of a new type, having the lower density and less heat capacity. Twelve cillion cubic yards of the fuel was extensively in London streets in 1770. Every of London was supposed to be heated by the introduction of the city streets in 1770.

The development of the motor was not complete the idea of the motor was not meant for the motor was not meant for the quantity of energy. It's varied and complex.

Pavement introduced in London was the first major implementation. The first major implementation in London was the first major implementation in America. The first major implementation in America was still in a country.
The removal of the tram tracks in
1968 was followed by the introduction of stone
platforms with a "kiln" of grade closed at
40mm, referred to as "stone blocks".

In the 1970's, the station layout was
changed and the platform length was
extended to 300m, with a new
structure. It was also reported that the
track was laid to a steeper angle at that time.
considered principle, claiming the form given in the latter case, the result is given.

The practicality of the test and the interpretation results renders this form of test

valuable.

Sec. 1. Method of Operation. Attempts have been made to evaluate energy absorption of aluminum alloys in the crushing principle, but the results have been unsatisfactory.

Fabrication - A device, originally, was

the aim of the test, the impact test, a direct test on aluminum and compositions.

The headed test specimen, 1-
to be placed on a plate and loaded to a
crusher with a spherical end of a cone indented through the body.

The critical point of the failure of the specimen occurred at the point of contact of the specimen to the hammer. A literal support is given to the apparatus, while a series of tests are conducted at temperatures of 95°C and 15°C.

Effecting impact by dropping a specimen from a particular height has not been found to be effective, while impact test extending the test for an additional time, is not usual practice.

A 9-f. square specimen was determined to be the most useful upon which a critical load may be determined.
Simulative tests - Simulative tests are made to test the actual constantly occurring material in the form in which the test is seen or encountered. They may be used to place the resistance to deformation.

Simulative, or service tests, are used. One of the first attempts to carry a normal test performed in the laboratory into the tests in the field is reported by Damon and Anderson, in that, although the test produces a number of advantages the validity of results obtained from it is good results from the former proved "mild" and the laboratory test or little value, while the laboratory test is not affecting the suitability of the material to service construction.

Damon reports the results of the laboratory and field regelial distances on the results given in the field and laboratory.

The materials are selected in order to the material. The selection of the materials is made with a view to forest action.
A small type of laboratory apparatus, particularly useful for \textit{force testing} a single specimen at a time. The test \textit{tube} is rigidly supported on a table, which is rotated at a constant speed by means of a solid rubber tyre wheel. The \textit{tube} is provided with a small contact surface at its centre, its deformation at the centre of the \textit{tube} under the influence of the \textit{force testing} provides an indication of the \textit{characteristics} of the material.

A static dynamic cumulative test was also carried out.
deterioration. This type of test can also show the susceptibility of the system to failure and apply limits to the system's performance. It should also demonstrate between normal traffic and climatic conditions.
A convenient means of measuring the behavior of the material under the action of traffic, with information which may occur when traffic is moving at a substantially lower or quite low velocities have shown little change under heavy traffic. This is probably in some cases due to the peculiarities of subsequent conditions are peculiar to specific to provide an absolute performance figure composition.

If surface deformation were to inappropriate compositions, the situation definitively was not, most likely in which the relative limit was not yet established. The report shows that many materials have a non-linear response to deformation, and that some materials exhibit a very small initial deformation. This is consistent with the low potential limitations of deformation.
To eliminate the need for an isolated test, a more rapid and efficient method of testing is required. The use of a cumulative test, such as the wheel-sink, would be the most practical for this purpose. These tests require a reasonably well-distributed test section and conditions can be controlled under banking conditions by a relatively short time.

laboratory tests, both in terms of the number of tests required and the time it would take to perform them. The results of these tests may be compared with the stress-strain curves to determine the suitability of the material for use in the construction of roads. The stress-strain curves can be obtained from compression tests conducted on the candidate materials under controlled conditions.
In view of the foregoing comments, it would appear that the properties of a bituminous macadam should be characteristic of the properties of a road material. Thus, conditions such as temperature and atmospheric pressure, in addition to the functional characteristics of the material itself, must be considered when testing the material.

It has been suggested that various properties of the road material, such as temperature and atmospheric pressure, should be taken into consideration when testing the material. This is because the properties of the material itself, such as temperature and atmospheric pressure, can affect the performance of the road material. Therefore, it is important to consider these factors when testing the material.
The results of the unconfined compression test are presented in a tabular form. The tabulated data includes the applied stress, the resulting strain, and the corresponding density of the material. The data is presented in a consistent format, allowing for easy comparison and analysis.

The stress-strain relationship is then plotted on a graph, with stress on the x-axis and strain on the y-axis. The resulting curve is a straight line, indicating a linear relationship between stress and strain. The slope of the line represents the modulus of elasticity of the material.

The density of the material is also calculated and presented in the tabular form. The density values are found to be consistent with the expected values, indicating that the material is of good quality.

The results of the test are then compared with the expected values, and it is found that the material performs well under the given conditions. The test results are also compared with similar materials to determine the relative performance of the material.

In conclusion, the unconfined compression test is a useful method for determining the mechanical properties of the material. The results of the test are presented in a clear and concise manner, allowing for easy interpretation and analysis.
The values are considered to be an indication of the practical suitability of the given concrete mix as a ratio of the stability to the maximum binder content.

Please refer to page 26 for more details. The ratio stability/flow and the ratio stability/blacktopping apparatus. These results are compared to a range of binder contents and mixtures of aggregates, rather than the performance of the individual binder content of a range of aggregates. However, since it is always desirable to use different compositions, the Marshall test may be considered in order of resistance to deformation or as the alternative.

Please refer, in discussing the application of the stability value to an assessment of resistance to cracking, with reference to one particular type of aggregate content. It can be stated that the stability value itself, provides a reliable measure of resistance to cracking. However, it is quite incorrect to make the general statement that the Marshall test is unsuitable for assessing resistance to cracking. The purpose of covering a range of binder contents is merely to select an optimum binder content for a particular aggregate mixture. The optimum binder content selected in this manner is one that, while providing the strongest mix with adequate workability and stability, is not too high, and is the closest possible to the measured optimum.

The Marshall stability, at the optimum binder content, of a range of aggregates have been shown to...
In a previous test program, the resistance of the materials of the mixture to be determined, was measured. However, in this test, a different method was used for that purpose. The equipment used was the Marshall isochronous test.

The Marshall test is commonly used in research reported in this thesis, and is especially popular for testing the resistance of different mixtures to test in this country. However, the test was not used at certain airfields in Britain, but was turned to road works. The equipment was modified, and a certain amount of experience had already been gained with the apparatus and in experimental techniques.
The physical properties of dense asphalt mixes.

Binder - The function of the binder is to maintain a durable and water-proof adhesive bond between the aggregate and asphalt particles. The binder content should be such that the pavement is covered with a ductile film of binder, keeping the aggregate particles together, and lending the pavement a rigid structure.

The optimum binder content providing the required durability has been defined by Keffer et al. which states the aggregate will tolerate without adverse effect sufficient in binder will result in a flexible, durable pavement, would be susceptible to gradual deterioration, unless an excess of binder would develop in the upper surface. Free and Hiltz have shown that asphalt concrete mixes, the binder could also act as a lubricant to the aggregate structure, leading to more resistance against the forces under heavy traffic.

In summary, we have stressed the importance of the correct binder-filler ratio for a stable and durable asphalt surface. By maintaining the correct binder content, we ensure the longevity of the binder, which has shown that the asphalt mix is capable of capturing internal forces and work with the pavement structure to improve performance. It will therefore be concluded that the correct binder content is essential for a
and the mixture.

It is of the utmost importance that the mixtures should be tested for their resistance to the various types of materials.

A brittle mixture will not withstand the stress of the traffic. However, the type of material will determine the resistance to deformation beneath the wheels. Neither Marshall stability nor Marshall flow is a sufficient test to determine the resistance to deformation beneath the wheels. Tests are carried out at equi-viscous temperatures to simulate the points of the binders.

Lauder and Teller have found that a linear relationship exists between Marshall stability and binder content for mixtures of binder and bitumen at optimum binder content. This relationship becomes more the binder content decreases and decreases again. This suggests that for optimum stability at high binder content and low binder content, the binder content becomes approximately the same.

The viscosity of the binder is affected by its temperature and the ambient temperature. This will result in a lot of problems with binders with unfavorable results. The binder type for the mixture is also important.
in rnxn

reduced lime stone, but some as,
were intended to increase the
simpler and reduce the stability.
That the substitution of
limestone has little effect on stability,
result in a reduction in strength, but 1
full scale conditions limits the validity of this report.

Two large groups of engineers have investigated this type of filler in bond asphalt.

Chalk, and chalky-lime fillers produce relatively high stability, whereas limestone, Portland cement, and gravel produce medium stability, and sand fillers produce low stability.

Of flow properties are roughly in the following
flow values at the higher stability. The
values are probably due to the fineness and shape of the filler, the
more irregular fillers accommodating greater proportions of sand, condition stiffer mixes of high stability.

Chen and Lee and Kirchen in a study of mixtures,
their tests on filler/binder compositions have

The increase in binder or asphalt content makes the mixture and the mixture composition dependent on the mineral source of the filler. The combined effect is illustrated by means of the bulk density of the binder composition. The specification is often referred to as the mixture source of the binder or the mineral source of the binder material. The mineral source is important, as it influences the mineral powder of the required composition.

Filling content is of vital importance to the physicochemical properties of the mixture. Tests on different fillers, such as sand, gravel, and crushed stone, show that, for a particular aggregate, there is an optimum filler content at which stability is minimum.

In the case of filled, unbound asphalt mixtures, bonding between the binder and the aggregate is important. Bonding between the binder and the aggregate is important and can be improved by using fillers. The amount of filler required can be optimized by determining the type of binder and the aggregate composition.

In summary, the stability of bituminous mixtures is determined by the increase of mineral content, which is relatively independent of filler type. The mineral source of the filler characteristics has been measured at the British and German associations. The British and German associations deal comprehensively with this aspect.
We are going to make a difference on the...
although it is not possible to advocate an ideal distribution, considerable work has been performed to determine the effect of aggregate grading on the stability of asphalt pavements, and there appears to be a general agreement.

Previous work by the author on hand asphalt mixture, for a continuously graded aggregate, the part which is retained has little effect on the magnitude of Marshall stability. The values of optimum binder content are, however, altered considerably with a higher content for the finer grading, as would be expected from considerations of aggregate packing and surface area. The plots were of a similar shape and with a similar drift in binder content. These findings are in accordance with Vallerga et al., who have found that the angle of friction of a dry uniformly graded granular material up to 0.2 inches in diameter is not affected by the particle size, or particle size distribution. Vallerga et al. has extended this work to cover the effect of gravel aggregate on the stability of asphalt paving mixes with similar results.

Graceo, Mayer, and the U.S. Corps of Engineers have
An elastic and resilient asphalt mixture is one which has the ability to return to its original condition after deformation. This ability is critical to the performance of the mixture in road construction, as it helps to ensure durability and long-term stability.

In the context of asphalt pavement, the inclusion of small quantities of mineral filler can significantly enhance the elastic properties of the mixture. The filler helps to bond the aggregate particles, improving the overall cohesion and reducing the likelihood of premature cracking and distress.

The research has shown that the optimal amount of mineral filler is based on the specific requirements of the project, including traffic conditions, climate, and subsurface characteristics. For instance, in areas with heavy traffic or severe weather conditions, a higher proportion of filler may be necessary to achieve the desired level of performance.

However, it should be noted that the use of mineral filler should be balanced with other considerations, such as cost, environmental impact, and long-term sustainability. Excessive use of filler can lead to increased costs and potential issues with material disposal.

In summary, the inclusion of small quantities of mineral filler can enhance the elastic properties of asphalt mixtures, improving their durability and performance. However, careful consideration is required to ensure that the benefits are maximized while minimizing any negative impacts.
The significance of the behaviour of asphalt mixtures for practical purposes, particularly asphalt surface mixtures and macadam, can be determined if the peculiarities of these materials and their interaction are studied in detail. For practical purposes, asphalt mixtures are predominantly used in their own as homogenous materials, for example, in continuous mixtures. However, the strength of the composite is not independent of the particles and shape of the material, as the mechanical interaction is greatly influenced by the "interaction between the particles" of the mixture.

Hartman and co-workers have found that the interaction is maximum to a rounded natural sand material in stability of asphalt, even when filled and sand asphalt. The results indicate that the stability of continuously graded sand asphalt mixtures with optimum binder content was found to increase from 8 to 12 as the rounded sand was gradually replaced by the more angular and ferro-carbon. Flow values ranged from 0.003 to 0.005, increasing more rapidly as the more angular sand became the more dominant ingredient. Illions indicated similar trends with the sand. It was found that a range of seven sands and nine combinations of seven gradations, were placed in order of stability by the "interaction of particle interaction, itself a function of shape and texture."
Nijboer\textsuperscript{32} page 93 et seq. has covered extensively and comprehensively the effect of the nature of the stone on the physical properties of dense asphalt mixtures and has found that, for a single sized stone in a sand/filler/binder matrix, the surface texture of the stone has a considerable influence on the angle of internal friction.

Hilliard\textsuperscript{116} makes it clear that "the unit of the porosity has been progressively reduced to a single aggregate", and is of the opinion that such a reduction indicates a greater degree of inherent stability in the cement.

If porosity of the aggregate becomes the inherent factor of its nature, then the stability of the mixture is directly related to the porosity of the aggregate. While results from this investigation indicated that a greater degree of stability is achieved in the continuous mix, but with respect to the stability of field asphalt mixes. The presence of coarse aggregates effectively reduced the possibility of high stability between binder and aggregate. The binder to the sand, and binder to the aggregate surface texture of the sand is the dominating factor in the tendency influencing the behaviour of the mix.

G. W. Niblett, \textsuperscript{156} has shown that the type of fine aggregate is of considerable bearing on the rate of deformation observed in a wheel tracking test. An increase in resistance with a corner, more angular sand has also been reported by Groome\textsuperscript{124} with a super-paved asphalt of 25 stone content, Pleace et al\textsuperscript{126} with a continuously graded dense tar mixture of 40% stone, ottman and Hiest\textsuperscript{131} with a continuously graded asphalt of 65% stone, and by Griffiths and Willans on a complete range of stone contents in continuously graded asphalt.
of the material. The size and shape of the stone was shown to have little effect on the viscosity of the mass, and this is supported by the findings of other investigators. But Mixon and Thornton, using a stone content of 30%, using four extreme shape of stone, have also found that the stone size has little effect on the stability of gap-graded asphalt at the same 30% of stone.

Abstract. Lee and Markwick have reported that the stone aggregates with a flaky aggregate offer a 50% greater fracture than cubical stone with similar surface characteristics. However, Grimes favours a cubical stone as it is less susceptible to fracture than flaky material of a similar size.

Stanton and Wyen have found that the surface character of the aggregate is the most important single factor of the aggregate affecting the stability of bituminous paving mixtures, and showed the use of rough stone to maintain as much interlock as possible between the aggregate particles.

Herrin and Boetz, Lottman and Boetz, and Lottman have concluded that shape and surface texture of the stone are the chief aggregate properties that contribute to the stability of asphalt mixes with a constant stone content and type, and that the size of the stone has little effect. The fact that the surface texture of the stone does have a limited effect on stability is also noted by Driffield and Haines, but these results should be interpreted with caution as the filler content used throughout this series of tests varied with the stone content.

Campbell and Siddall claim to have conclusively demonstrated that texture, not shape, is the dominating characteristic of the coarse aggregate, and that the shape and texture of the fine aggregate are significant factors affecting the stability of asphalt mixtures.
C. Assessment of particle size, shape and texture.

1) Particle size - Although the work reported in this thesis is not directly concerned with the measurement of particle size, it is felt that a brief review of methods of particle size analysis is necessary when interpreting results of assessment of particle size and texture. This is especially so since it is often impossible to isolate each of the three surface characteristics.

The size of a spherical particle is defined completely by the diameter, but the "size" of an irregular or non-uniform particle is dependent upon the property under consideration, and the method of measurement. Various methods of particle size measurement, such as: volume, surface area, resistance to motion in a fluid, or light scattering properties may be compared with the diameter of a sphere of equivalent characteristics. An assessment of size based on the ratio of one of these properties to that of a sphere will produce different values of size depending upon the method used, even if the sphere will possess the same size by different methods.

The three basic methods of size measurement are defined as:
- diameters of the spheres of equivalent volume
- surface area
- resistance to motion in a fluid, and termed the "volume diameter" - \( V \), the "surface diameter" - \( S \), and the "drag diameter" - \( D \), respectively.

Heywood has divided the diameters of irregularly shaped particles into two main categories: those based on the size of the projected image of the particle placed in its most stable position, and those based on the volume of the particle.

The size of a particle as determined from its projected image has been defined by Heywood as the diameter of a circle of equal area, and termed the "mean projected diameter". Partin derives the mean projected diameter from the average length of a pair of straight lines each bisecting the area of the projected image at right angles,
1eret takes the average of several points of measurements of the overall size of the projected image taken at right angles. Heywood reports "errors" of $-\frac{1}{3}$ and $+\frac{1}{11}$ in Martin’s and Feret’s methods respectively, based on his "true" values.

Heywood’s method of particle size measurement has been adopted by the British Standards Institution where the projected area is matched against a series of circles on a graticule, the diameter of the series being $\sqrt{2}$ progression, and the statistical accuracy of this method is discussed by Fairs.

The volume diameter of a particle is taken as the cube root of volume, and is smaller than the mean projected diameter. The volume diameter, attributed to Andreasen, is suitable for which are large enough for a weighable quantity to be counted.

Tickell has defined the size of a particle as:

$$d_{vol} = \sqrt{\frac{6}{\pi} \cdot \frac{V}{A}}$$

where $l$, $b$ and $t$ are the length, breadth and thickness respectively.

A comprehensive review of methods of particle size measurement is presented by Hawksley, who, with Irani and Callis, has dealt with sedimentation and elutriation techniques in considerable detail, although remarks in both papers are confined mainly to particles in the sub-sieve range.

In classifying particle size by sieving, the grain shape has a considerable influence on the passage of the particles through the sieve. It is generally considered that sieving separates the particles roughly according to their volume, but Lees expresses doubts as to the validity of this assumption, although the extreme cases quoted by Lees are rather outside the particle shapes commonly used in civil engineering and, more particularly, in road-making aggregates.

However, for the purposes of this research, the particle size
has been defined as the root mean square of the
in a quarter-size progression.

11. **Particle shape** - The relationships between the various
of a particle are dependant upon its shape, and non-dimensional
combinations of these "sizes" may be employed to assign various
values of "shape factor" to the particle. The choice of "sizes"
will depend upon the purpose for which the shape factor is required.

Discussion on shape factors have been concerned mainly with
the problem of converting the projected dimensions of the particle
into volume or surface diameters but, since the innovation of
permeability techniques measuring directly the specific surface
diameter, the topic has been somewhat neglected.

Shape has been defined by Heywood\textsuperscript{129} as being "the closer
with which the particle approximates to certain geometrical solid
such as ellipsoids, prisms or tetrahedra", and shape in general
referred to certain properties of a sphere, either in its
dimensional state or in the form of its projected image.

Heywood's system of shape factors\textsuperscript{136} is based on the two
projected diameters, or "area diameter" - \( d_a \) and the "perimeter
diameter" - \( d_p \), producing two useful shape factors:

\[
\frac{d_a}{d_p} \quad \text{and} \quad \frac{\delta^2}{\Delta^2} \quad \text{(these symbols are defined in the previous section)}
\]

Heywood\textsuperscript{136} defines the "volume constant" - \( k \) as the ratio of
the volume of the particle to the cube of the "area diameter" i.e.
\[
k = \frac{1}{2} \frac{\delta^3}{d_a^3}
\]

Similarly, the "surface constant" - \( \tau = \frac{1}{2} \frac{\Delta}{d_a} \), and
the "contour ratio" - \( \frac{d_p}{d_a} \). From a series of measurements of

65.
volume and surface area of large stones, Heywood has obtained empirical relationships between the contour ratio and the volume and surface constants, enabling the surface constant of a fine particle to be evaluated from a microscope examination. This surface constant may be used either to produce a shape factor, or evaluate directly to give the surface area of the particle.

Tickell also working with projected areas, defined the shape of a particle by two parameters, "roundness" and "sphericity":

\[
\text{roundness} = \frac{\text{projected area}}{\text{area of smallest circumscribing circle}}
\]

\[
\text{sphericity} = \left( \frac{\text{diameter of inscribed circle}}{\text{diameter of circumscribing circle}} \right)^3
\]

while Wedell defines "sphericity" as:

\[
\text{sphericity} = \frac{\text{surface area of sphere of equivalent volume}}{\text{surface area of particle}}
\]

These definitions of roundness and sphericity were evolved originally for use in geology to define the wearing of sedimentary rock fragments, the base sphere being the largest sphere which could be produced when wearing down the particles without fracture. Roundness would not appear to be a good parameter in the assessment of particle shape, as it does not take account of either the regularity or the rugosity of the surface, while sphericity requires a further restricting parameter to define shape completely, as different shaped bodies may possess equal sphericities.

It is not intended to pursue the problem of assigning shape factors any further, and the reader is referred to papers by Lees and Frederick for further reading.
The assessment of shape and equivalent spherical diameter of larger particles have been investigated by Tester and Hapgood, and Liepelt who have measured the leading dimensions of the particles by means of calipers and gauges, but their work was essentially confined to coarse and very coarse sand sizes.

Harwick has defined the state of a particle by two parameters, the relative roundness (or form) and the relative degree of elongation (or shape), and his methods of determination of roundness and elongation are now included in I.S. (12).

The measurement of larger particles in terms of surface area has been performed by Goldbeck who wraps the stone particles in 1/20 sheet lead, by Truesdale et al and Fox who employed similar techniques, and by Hedlin and Collins who measured the surface area by attaching a layer of nickel powder to the stone. The surface area in all cases is measured from the weight of the layer of nickel powder to the stone.

The procedures discussed so far to assess the shape of aggregates have been confined entirely to the study of individual particles. This method of analysis is often tedious, restrictive in the size of particles studied, and may lead to errors in the random selection of representative samples.

The difficulty in studying the properties of individual grains has led to the development of a more rapid and more realistic measure of the combined effect of aggregate shape and surface texture in bulk.

Schiell has assessed the shape of single sized aggregate particles by comparing the difference between the amounts of the sample passed by square- and round-hole sieves of the same nominal aperture size. This procedure has been found to be extremely
susceptible to slight changes in particle size distribution, and is limited in its application to the study of the more extreme degrees of particle shape.

Shergold has developed a measure of angularity in which the proportion of voids in a sample of single sized aggregate particles, compacted in a standard manner, is recorded and reduced to an 'angularity number'. In order to assign a value of angularity it was found that a well-rounded gravel compacted to a void content slightly in excess of 33%, and thus 33% was taken as the minimum void content possible with natural aggregates, and was used to produce an angularity number "N" where \( N = \text{percentage voids} - 33 \). This method proceeds the use of spheres as a datum as it was found that spheres compacted to a minimum void content of 41%, while 3-inch cubes produced a minimum void content of 30%. The test is therefore restricted in its application to non-regularly shaped bodies.

Shergold's angularity test is now included in the third revision, and subsequent editions of A.S. 812, and has been used by Henrieff to study the effect of grading and shape on the bulk density of concrete aggregates.

Hosking has found that the application of an equal compactive effort to samples of aggregate tested in different sized containers, results in a lower void content being achieved with a smaller container. This is due to the relatively greater degree of compaction applied to the aggregate particles, and Hosking advocates a reduction in compaction roughly in proportion to the reduction in the container size. This recommendation is in accordance with that found in an appendix to the method of determination of angularity number in A.S. 812.

The measurement of voids in compacted aggregate has also been
employed by Buenz\textsuperscript{156} to assign a value of "particle index" to the geometrical characteristics of the aggregate. The single sand aggregate is compacted in a standard manner in a rhombohedral box, and the void contents of the aggregate at two degrees of compaction is evaluated in "particle index" $I_a$ of the sample where

$$I_a = 1.25V_{10} - 0.25V_{50} - A$$

where $V_{10}$ = percentage voids at 10 compactive strokes/layer,

$V_{50}$ = percentage voids at 50 compactive strokes/layer,

$A$ is a constant, dependent upon the size of the aggregate, based on the packing characteristics of polished aluminium spheres.

In the light of Shergold's comments on the behaviour of particles of regular geometric shape, and the experimental results discussed later in this thesis, the validity of employing the packing characteristics of spheres in assigning a value of "particle index" to non-regular aggregate particles would merit further investigation.

A final method of assessment of particle shape in bulk is based on the measurement of the specific surface of the fine aggregate by means of the rate of flow of a fluid through an uncompacted bed of randomly packed particles.

Early work by Carmen\textsuperscript{157,158} and Lea and Nurse\textsuperscript{152} is based on the principles of permeability laid down by Honer\textsuperscript{159}, but is confined to the testing of fine powders in the size range of 2 to 100 $\mu$m. A comprehensive review of methods of surface area measurement has been presented by Conner\textsuperscript{161,162}, and although the procedures described are basically designed for the analysis of fine particles, certain methods such as dye adsorption, permeability and photo-extinction may
It will be appreciated that it is often impossible to apply the basic principle of surface area measurement for fine particles to sand. In the ungrafted case, the specific surface (cm²/cc) is inversely proportional to the particle size, so that an increase in particle size results in a considerable decrease in specific surface.

The possibility of fluids through granular beds has been analysed by von Buol and Johnson. Shacklock and Walker have employed the principle of permeability to measure the specific surface of cementitious material and aggregates, with a view to relating the specific surface of the aggregate to the workability of concrete mixes. A satisfactory correlation was achieved for mixes of a fine aggregate ranging, but with greater readings, the relationship became invalid, relating to the surface area of the aggregate.

The permeability technique has also been applied by later workers, and it is evident that the permeability test procedure is too lengthy for a quick assessment of particle size.

(iii) Particle surface texture - The surface texture of sand has long been recognised as a major factor governing the degree of particle interaction, and may well be an important property in determining the potential adhesion of a binder to its surface. The term "surface texture" has been employed to include both the microtopography and the permeability of the aggregate surface.

While the majority of sand aggregates are hardly affected by absorbed moisture, a few, while possessing a high stability when dry, are capable of absorbing sufficient moisture to become extremely...
unstable when wet.

Pettier et al.\textsuperscript{167} have found that the capacity for capillary flow of a given stone is related to the physical, rather than the chemical, composition of the mineral, since there exist both absorbent and non-absorbent examples of calcite, dolomite and quartz. The physical mechanism of absorption is explained by the passage of moisture through the intercrystalline channels of the aggregate.

While it would be expected that binder adhesion would be related to the absorptive properties of the aggregate surface, Pettier\textsuperscript{167} at the University of Birmingham has shown that binder adheres more dependent upon both the petrography and the microtopography of the aggregate surface, supporting the suggestion by Knight\textsuperscript{170} that hydrophilic stones produce a greater affinity to binder than hydrophobic stones.

The surface texture of solid borosilicate glasses related to various degrees of "surface roughness", has been found by Pettier et al.\textsuperscript{170} to have a direct bearing on the strength of the aggregate material, indicating that the surface texture of the aggregate contributes jointly with the shape in producing the strength of mixture.

A quantitative assessment of surface texture is difficult, although several methods have been developed, none has so far been generally accepted, or widely used. Various methods include the study of the trace of a stylus drawn over the surface in mechanical, electrical or optical instruments, a visual examination of the surface either in an enlarged image or of the profile of a coating, or by the examination of the trace illuminated by oblique lighting from a slit source, parallel to the surface of the aggregate.

Knight\textsuperscript{170} has developed a method of measuring the surface texture.
texture of stone samples in which a number of pieces of 1-inch aggregate are placed in a 6-inch test-tube, 1-inch in diameter, and surrounded by resin. The test-tube is broken away, and cross-sections, 0.1-inches thick are cut through the aggregate at various planes perpendicular to the axis of the cylinder. The sections, mounted on a slide and ground to a thickness of 0.001-inch, are examined under a projection microscope. The perimeter of each image at a magnification of 125 is traced, and the length of the profile, measured by means of a map-measuring wheel, is compared with the lengths of various "unevenness" lines drawn as a series of chords doubling in length from 0.5 cm. to 5 cm. The difference between the "unevenness" line and the sample length is expressed as a proportion of the sample length to provide a value of "roughness factor". The test is limited in its application to the larger aggregate particles, and results are liable to variation depending upon the positioning of the cross sections, and the portion of profile studied.

Apart from illustrating the importance of surface texture, both to adhesion and particle interaction, it is felt that an assessment of surface texture is of less significance than a combined measure of aggregate shape and surface texture when investigating the effect of the surface characteristics of aggregate on the properties of an aggregate mixture.

D. Assessment of the combined effect of aggregate shape and texture by its rate of discharge from an orifice.

The rate of discharge of granular materials from storage bins and hoppers varies widely with the shape of the container and also with the characteristics of the material. The size of materials
stored may range from cement and fertilizers to coal and iron ore, and the capacity of the hopper from a few pounds to many thousands of tons.

The design of an effective storage hopper is dependent upon a thorough knowledge of the behaviour of the material in bulk, and it is widely recognised that the characteristics of the granular material have a considerable effect on the rate of discharge of the material from the hopper. Aggregate characteristics such as density, grading, shape, texture and cohesion, all contribute to the final rate of discharge, and it is evident that with non-cohesive material of similar density and grading the shape and texture alone have a considerable bearing on the rate of discharge.

The phenomenon that smooth textured, rounded sand particles offer less resistance to free flow than do rough textured angular particles, has been employed by Rex and Peck \(^{171}\) to assess the ease of particle interaction of various types of sands used in situ, in flexible pavements.

Rex and Peck record the time taken for 500 gr. of a single sized aggregate sample to flow through a circular orifice of 1-inch in diameter and express the result as the "flow rate" in seconds per 100 cc. A "time index" is taken as the ratio of the flow rate of the sample to that of a rounded (Ottawa) sand, and is a measure of the combined effect of "relative angularity and surface roughness". Rex and Peck specify a single sized sand fraction passing No. 10 and retained No. 25 B.S. sieves, probably as this size is large enough to be free from any possible cohesive effects, or small enough to ensure free flow from the orifice with the most angular and roughly textured material likely to be encountered. Specific gravity tests are easily performed on sands of this size range.
Gloem and Byner have used the efflux test to determine the "frictional character" of sands used in concrete research. They have found that the frictional character of the fine aggregate greatly influences the water/cement ratio required for a predicted compactivity, which is related to the efflux value. Gloem and Byner have specified the use of three sand sizes blended in equal quantities, 1/4 - 1/2 and 15-50 U.S. sieve sizes, and employ a 6-inch diameter cylinder. They have also collected the discharged sand in a graduated cylinder and related the loose and compacted bulk densities to the efflux rate.

Detailed tests on the rate of efflux of both single-sized and graded sands from circular orifices has been reported by Iseki, who has found the test to be both quick and effective in predicting the relative order of increasing particle interaction of a number of concrete mixes.
CHAPTER 5

SCOPE OF INVESTIGATION

It is apparent from the study of previous work that the grading, shape and texture of the aggregate has a considerable influence on the characteristics of dense bituminous mixtures. However, no evidence has been found of an attempt to assign a numerical value to the degree of particle interaction of an aggregate in order that the effect of particle interaction on the behaviour of an asphalt mixture might be studied in a quantitative manner.

Work was therefore undertaken to develop a technique whereby the degree of particle interaction of a range of aggregates could be assessed numerically, and, if possible, to achieve a correlation between these values and their influence on the physical properties of dense asphalt mixtures.

Marshall test curves and effect of aggregate type

Although the Marshall test procedure has been employed on a number of occasions in the study of the properties of hot bitumen, this work appears to have been confined to a range of binder content in the immediate vicinity of the optimum binder content. The general relationship is that with increasing binder content, the stability value rises to a maximum and then decreases, behaviour that is established by Davenport in the study of the compaction of soils. However, previous work by the author suggested that a minor stability peak is apparent under certain conditions at a binder content somewhat below the peak normally observed.

In consequence, it was felt necessary early in the investigation to establish the shapes of the various Marshall test curves with great
care, and to study the characteristics implied by their form.

To this end a large number of specimens was produced, with close control in the preparation and testing, with a constant aggregate grading and filler content, and with the widest practicable range of binder content.

The aggregate used for this purpose was a crushed granite, selected for its apparent uniformity of shape and surface texture throughout the range of sizes required. The supply of aggregate was converted to single sizes and recombined to give a continuous grading with 5% stone (defined as that aggregate retained on a No. 7 R.S. sieve) of a 2-inch maximum size. Crushed limestone dust (nominally 100 passing a No. 200 R.S. sieve) was used as the filler, a filler content of 2% by weight of the aggregate (including filler) being used throughout this section of the work. The binder used was a 60/70 penetration straight run bitumen and binder contents, expressed as a percentage, by weight, of the total mix, were varied in 2% increments over the range of workable mixtures.

A similar range of binder contents was examined using an irregular but smooth-textured flint gravel, blended to an identical aggregate grading and with the same filler content. It was intended to produce mixes with an aggregate of as different a nature to the crushed granite as possible, i.e. well rounded and smooth-textured. However, the irregular flint gravel was readily available in sufficient quantity and was of sufficient contrast to the granite to provide a reasonable assessment of the influence of particle shape and texture.

Two further sets of results were obtained by interchanging the stone and sand fractions of the crushed granite and the irregular gravel, in order that the relative contribution of the nature of the stone and sand to the behaviour of dense asphalt mixes could be.
Assessed. Aggregate grading and filler content were again kept constant, in order that the effect of aggregate shape and texture be studied independently of other variables.

**Effect of aggregate grading**

The effect of altering the aggregate grading was next studied, with aggregate type and filler content remaining constant. The crushed granite was again adopted for the reasons given previously, but especially as it was considered that any change in aggregate shape or texture with size would be accentuated with change in grading.

Four continuous gradings of constant maximum size, and filler content of 7%, were chosen, namely 30%, 40%, 50%, and 60% stone content. Data relating to the 50% stone content grading was available from the previous section of work.

The results of tests on the mixes have been related to the fineness of the various aggregate gradings, enabling an optimum filler content to be chosen. The value of this figure is, of course, limited in its practical application since it applies only to the aggregate type and grading, filler and binder used in this investigation.

**Relative behaviour of continuous and gap gradings**

A limited range of continuously graded aggregate material, with the 7-20 sand fraction omitted, was produced with a natural granite stone and irregular Flint gravel void. Further mixtures were produced of a more pronounced gap-graded nature consisting of a single sized crushed granite stone, and a naturally well rounded "angular" sand, mainly passing No.52 and retained on No.10 sieves.

No graphical comparison of these results is possible, but findings are discussed.
Effect of particle interaction

In order to investigate the effect of particle interaction on the behaviour of asphalt mixtures, it was necessary to procure a large variety of aggregates for comparison. It was decided to restrict the investigation to sand asphalt mixes since these are potentially more homogeneous than stone filled asphalts, leading to better reproducibility of results, and further that a wide variety of sand types was readily available. This was justified by the review of previous work which indicates that the sand fraction of a stone filled asphalt is the dominant factor influencing the behaviour of the mixture.

In order to assign a numerical value to the degree of particle interaction, it was decided to employ the technique of measuring the rate of efflux of the aggregate particles from a circular orifice. The apparatus developed by Bowing, was employed for this purpose.

In the study of the rates of discharge of granular materials from a circular orifice tests were first made using ballotini - nominally spherical glass particles of various sizes grades, with the 1, 2 and 3-inch diameter orifices. The effect of particle shape and texture was then investigated using a variety of crushed rock and natural sand.

An expression has been derived for calculating the equivalent particle diameter of a graded aggregate, and the packing characteristics and efflux rates of various single sized and graded - graded and sand combinations have been investigated.

A range of sand asphalts was finally produced, each blended to an identical aggregate grading, and with a crushed limestone filler content of 10%. The binder was the same type as that used in the
previous investigations.

A full range of binder contents was first investigated with
the crushed granite aggregate, again over the widest practicable
range of binder contents, to establish as well as possible the shape
of the various Marshall test curves for sand asphalt mixes.

Eleven other sands and sand combination asphalts were finally
produced in a range of binder contents about their predicted
optimums. Various plots of dry and mixed aggregate characteristics
enabled a basic understanding of the significance of particle inter-
action and the stability of asphalt mixes to be achieved.

Footnote

For convenience, work on the efflux test apparatus will be
dealt with in full, prior to the discussion of results of asphalt
tests.
Summary of work in thesis

Phase 1
Stone filled asphalt
Continuous grading - 50% stone
7% limestone filler

<table>
<thead>
<tr>
<th>Stone</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>granite</td>
</tr>
<tr>
<td>1b</td>
<td>gravel</td>
</tr>
<tr>
<td>1c</td>
<td>granite</td>
</tr>
<tr>
<td>1d</td>
<td>gravel</td>
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</tbody>
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Phase 2
Stone filled asphalt
Continuous grading
Granite aggregate throughout
7% limestone filler

<table>
<thead>
<tr>
<th>2a</th>
<th>30% stone</th>
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</thead>
<tbody>
<tr>
<td>2b</td>
<td>40% stone</td>
</tr>
<tr>
<td>2c</td>
<td>50% stone</td>
</tr>
<tr>
<td>2d</td>
<td>60% stone</td>
</tr>
</tbody>
</table>

Phase 3
Stone filled asphalt
7% limestone filler

3a Granite stone, gravel sand continuous grading, 7-25 sand fraction omitted.
3b granite stone, rounded asphalt sand, gap grading (1/2" stone, mainly 50-100 sand)

Phase 4
Analysis of rate of discharge of material from circular orifices, tests on single sized and graded sands.

Phase 5
Sand asphalts
Continuous grading
1% limestone filler
Various aggregate types.
### MATERIALS USED

**Binder**
- Shell "Naphath" 60/70 pen. straight run bitumen
- Measured penetration (average) 59

**Filler**
- Crushed limestone nominally 100 passing No.200 mesh T.I. slope
- British quarries Co. Ltd., Borough Green quarry, Orpington, Kent.
- Gritty Dolomitic Limestone (L)

**Aggregates**
- Crushed granodiorite (soda-granite) (G)
- Thames Valley Gravel - Han River Grit Co. Ltd., Stains Lane Pit, Nr. Chertsey, Surrey.
- Irregular flint gravel (A)
- Ballotini - English Glass Co. Ltd., Leicester.
- Nominally spherical glass spheres (+)
- Grove Granite - Grove Granite Co. Ltd., Grove quarry, Nr. Leicester.
- Granophyric diorite (F)
- Harden Cr. - Limmer and Trinidad Lake Asphalt Co. Ltd., Harden Redstone quarry, Biddulph, North. Albite - biotite - porphyrite (P)
- Tyroscene - andesite (A)
- Leighton Buzzard - George Cainside (Sand) Ltd., Munday's Hill Pit, Heath and Mecn, Nr. Leighton Buzzard, Beds.
- Irregular white silica sand (A)
- Limestone - Amalgamated Roadstone Corporation Ltd., Arnold quarry, Chipping Campden, Glos.
- Dolomitic limestone (L)
Aggregates (continued)

Enderby C.F. - Enderby and Stoney Stanton Granite Co. Ltd.,
Enderby Mill Warren Quarry, Enderby, Leics.
Quartz - diorite - porphyrite (F)

St. Ives Gravel - St. Ives Sand & Gravel Co. Ltd.,
Church Farm Pit, St. Ives, Hunts.
Rounded flint and quartz gravel (A)

Leighton Buzzard Rounded Silica - George Garside (Sand) Ltd.,
Double Arches & Churchways Pits,
Heath and Reach, Nr. Leighton Buzzard, Beds.
Rounded golden brown silica sand (A)

"Asphalt Sand" - St. Ives Sand & Gravel Co. Ltd.,
West Malling Pit, Wrotham, Kent,
Fine rounded flint gravel (A)

+ Artificial Trade Group
A Gravels
G Granite Trade Group
L Limestone " "
P Porphyry " "
B Basalt " "

82.
### AGGREGATE GRADINGS

<table>
<thead>
<tr>
<th>2.3 sieve size</th>
<th>30% stone</th>
<th>40% stone</th>
<th>50% stone</th>
<th>60% stone</th>
<th>Gap</th>
<th>Sand</th>
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Aggregates were screened on a quarter size progression of sieves and blended to the gradings shown above.

The intermediate half size progression of sieves, i.e. 1", 2", 3". No. 10, 18, 36, 72 and 150 were used to reduce batching errors in the main sieve sizes shown above.
Plate 1

TYPICAL 3" - 4" STONES USED IN THE INVESTIGATION
Mountsorrel crushed granodiorite

Shepton Mallet basalt

Thames Valley irregular flint gravel

St. Ives rounded flint gravel

Ballotini nominally spherical glass particles

Plate 2

EXAMPLES OF (7-10) SIZE FRACTION OF SANDS USED IN THE INVESTIGATION
CHAPTER 6

ASPHALT TESTS - EXPERIMENTAL PROCEDURE AND TECHNIQUES

The preparation of specimens and experimental procedure for the testing of both stone filled and sand asphalts, were based on the standard Marshall test procedure as set out by the Marshall Consulting and Testing Laboratory
9, the Air Ministry Specification, No. 201
10, and the Road Research Laboratory
173. Certain variations, however, were introduced as a result of preliminary tests carried out by the author in an earlier investigation
119.

The Marshall test procedure is prone to considerable experimental error and, in consequence, specification requirements call for a minimum of 4 and preferably 8 specimens at each condition during the course of routine mix design. In order to ensure the maximum reproducibility of results, and to reduce certain experimental errors to a minimum, batching and temperature limits permitted by the specifications have been reduced. The procedure followed is set out below.

Preparation of specimens

The blended aggregate is heated to a temperature of 325°F in a flat bottomed mixing bowl, and the correct amount of binder at 300°F weighed in. The mix is then thoroughly stirred with a wooden spoon until all the aggregate surface is fully coated with binder, a pre-weighed amount of filler added cold, and the mix again stirred until no trace of filler remains visible.

A test mould, heated for a period of some 10 minutes at 300°F, is placed on a balance platform, an asbestos sheet being provided to avoid damage to the weighing mechanism, and a stiff paper disc placed
into the mould to prevent adhesion of the material to the base.

The predetermined weight of mixed material is weighed into the mould, the bottom half inch of loose material first being lightly compacted with the wooden spoon, before the rest of the material is added. The remaining material is returned in the covered mixing bowl to a low gas, to reduce heat losses while awaiting use.

The surface of the material in the mould is then subjected to 15 blows of the wooden spoon, and the surface moulded to a dome shape in order that the initial compactive effort might be directed evenly throughout the specimen.

When the temperature of the specimen falls to 275°F, as indicated by a pocket "Rototherm" placed in the mixture, a second paper disc is placed on the specimen face and the specimen compacted.

Compaction is effected by a weight of 10 lb. being dropped 75 times from a height of 18-inches to the surface of the specimen. This is applied to both ends of the specimen, an automatic compactor shown in plate 1, being used in order to reduce operator variable. The detachable compacting head is heated for a few minutes prior to its contact with the first face compacted, in order to avoid chilling the surface of the mixture.

After compaction, the specimen and collar are cooled in a tank of circulating cold water for a period of some 2 - 4 minutes, and the specimen extruded by means of a hydraulic jack.

The specimen is finally checked for height, and placed on one flat face where it is left to cool and dry out.

While the specimen and collar are cooling off in the water tank, the remaining portion of the mixed material is utilized to produce a second specimen in the manner described above.
Plate 3

THE AUTOMATIC COMPACTOR
Plate 4

THE MARSHALL TEST APPARATUS
Testing of specimens

After a 24 hour period of drying out, each specimen is tested for specific gravity by determining the weight in air and in water, as set out in B.S. 812. This value, together with a knowledge of the specific gravity of the constituent materials and mix proportions, enables the void characteristics to be calculated. A theoretical analysis is set out in appendix 6.

Specimens are placed in pairs, at 5-minute intervals, into a water bath operating at 60°C, for a period of 45 minutes prior to testing to destruction. The breaking head of the Marshall testing apparatus is also placed into the water bath for some 10 minutes prior to testing of the first pair of specimens, and after the testing of each pair of specimens in order to avoid any chilling of the specimen during testing.

Specimens are tested individually, each in turn being placed into the heated breaking head, positioned under the load-transmitting proving ring of the testing apparatus, and subjected to an axial rate of strain of 2-inches per minute. The maximum load sustained by the specimen and the deformation undergone in attaining this position, are recorded as the "stability" and "flow" values respectively of the specimen.

EXPERIMENTAL TECHNIQUES

In order to reduce experimental errors to a minimum great care was exercised to ensure that the procedures selected were consistently adhered to, both in the preparation and testing of specimens.

Twelve pairs of specimens were produced in each mixing session,
taking approximately 20 minutes to manufacture each pair. The considerable length of time taken to ensure that all relevant temperatures and operations were ready at the correct time precluded any break in the production routine.

a) Preparation of constituent materials

i) Aggregate - The clean dry aggregate was screened to single sizes on a quarter size progression of B.S. sieves, and stored in air-tight drums.

When required for production, the various aggregate sizes were blended to the maximum accuracy permitted by the particle weight. This was found to be within the order of ± 1 gm. for \( \frac{1}{4} \) - \( \frac{1}{8} \)-inch stone, and within balance accuracy for aggregates less than \( \frac{3}{16} \)-inch in size.

Aggregates were blended in twelve batches of 2500 gm. each, less the weight of the filler which was added to the material during mixing. Each batch of aggregate was sufficient for the production of one pair of specimens.

The aggregate was placed in an oven, and retained at a temperature of 325°F over night, some 18 hours before being required for specimen production.

ii) Filler - The predetermined weight of filler was weighed out into each of twelve small tins, each tin containing the appropriate amount of filler for one batch.

iii) Binder - When required for use, the binder was heated to a temperature just sufficient to permit it to be transferred from the medium sized tins in which it is stored to small tins of approximately 500 cc. capacity. These small tins of binder were heated to the required temperature of 300°F not more than one hour prior to use, in order to minimise the possibility of a reduction in bitumen penetration.
resulting from prolonged heating. None of the tins of binder was reheated more than once, as severe hardening of the bitumen, resulting in a considerable drop in penetration, is likely under repeated reheating over a wide temperature range.

b) Preparation of specimens

When required for the production of a pair of specimens, the heated aggregate was transferred from the oven to the mixing bowl, and placed on a gas ring where it was thoroughly mixed and retained at a temperature of $325 \pm 5^\circ F$.

i) Mixing - Previous experience has indicated that hand mixing of the material is preferable to machine mixing, as a more thorough distribution of binder and filler is achieved in a shorter time, thus reducing heat losses, and also providing better control over the mixing operation, especially with stone filled mixes.

The filler was added cold during mixing in order to follow normal working practice, and to reduce the problem of dusting.

ii) Discs - Stiff paper discs were employed to eliminate adhesion of the mixed material to the mould base or compacting head, and in preference to the filter papers more generally used, as they were found easier to remove after compaction, and could be used several times thus reducing the labour involved in trimming 10 cm. diameter filter papers to the required size.

iii) Weighing-in - Experience and previous results enabled the order of specific gravity of the compacted specimen to be predicted, enabling the appropriate weight of mixed material to be placed in the mould, resulting in a compacted specimen of the correct height of $2\frac{1}{2} \pm 1/16$-inch. This precaution reduces the error involved in applying a correction factor to the stability values of specimens.
falling outside the stipulated height range.

iv) Compaction - Compaction was carried out at an indicated temperature of 275°F, never at a higher temperature, and specimens whose temperature fell below 270°F before compaction were rejected. The portion of mixed material that was kept warm while the first specimen of the pair was being compacted was retained at a temperature of between 280°F and 300°F, but not reheated, and any mix falling outside this range was rejected.

v) Cooling and extrusion of specimen - The specimen and collar were cooled for a period of some 2 - 4 minutes and extracted by means of a hydraulic jack. No advantage was gained in extending the cooling period in order to facilitate extraction; in fact, specimens were found more difficult to extract after further cooling time due to the contraction of the collar onto the specimen.

vi) Drying out of specimen - Complete cooling and drying out of dense asphalt specimens has been shown by demonstration samples to be achieved in less than one hour, further time being required only for the more permeable specimens at very low binder contents.

c) Testing of specimens

i) Specific gravity tests - In performing specific gravity tests, weighings were executed to within \( \pm \) 1 gm., and specimens weighed in water as quickly as possible, as the slight porous nature of some specimens resulted in an apparent increase in the submerged weight as the surface of the specimen absorbed a little water.

ii) Marshall test - Specimens were heated for the test at a temperature of 60 \( \pm \) 1°C for a period of 45 \( \pm \) 1 minutes, and transferred as quickly as possible to the testing head in order to reduce heat losses to a minimum.
Testing was simplified by the use of a non-return dial gauge for flow measurements, necessitating the reading of only one dial gauge on the proving ring during the short period of the actual test.

The above precautions enabled specimens to be produced to within an accuracy of stability of about 200 lb., but the inherent errors in the test as a whole did not warrant an accuracy in stability and flow readings of more than 5 lb. or 0.01-inches respectively.
CHAPTER 7
THE EFFLUX TEST

a. Description of apparatus

The apparatus used to measure the rate of discharge of granular material from an orifice is shown in plate 3. It consists of a 5-inch diameter container, 8-inch deep, surmounting a tapered section of a slope of 30° to the vertical axis. Interchangeable orifice units permit an orifice of 1/2, 5/8 or 3/8-inch nominal diameter to be installed while retaining the constant drain angle.

The drain angle of the apparatus is in excess of the largest "angle of repose" of material likely to be encountered ensuring full plug flow, rather than core flow, of the discharging material. Horizontal cross sections throughout the apparatus are circular, thus eliminating the problem of nip exerted by corners and avoiding any stagnant pockets in the material.

Discharging material is collected in a container 4-inches in diameter and 5-inches deep, the bottom of the container being between 7 and 8-inches below the orifice, depending upon the size of the orifice in use. The upper edge of the container is provided with a 1-inch lip to facilitate the striking-off of surplus material.

The apparatus is retained rigidly on a 1-inch base plate, 9-inches by 11-inches. Three 5/8-inch diameter vertical rods support the apparatus and it may be levelled by means of four foot screws.

b. Experimental procedure

A specified weight of material is placed in the apparatus with the orifice blocked off. The orifice is unblocked and the time taken for the material to discharge itself recorded to within 0.1 seconds.
Plate 5

THE EFFLUX TEST APPARATUS SHOWING
THE 1/8", 3/8" AND 1/4" DIAMETER ORIFICES
Excess material in the receiving container is struck off with a straight edge, and the weight of the container and loose material recorded. The material is then compacted in a standard manner, and the weight of compacted material and container ascertained.

With a knowledge of the density of the particles under examination, the efflux rate and the proportions of voids in the loose and compacted material are subsequently determined.

c. **Experimental techniques**

It was found that by blocking off the orifice with the operator's index finger, the material could be released and the stop watch started simultaneously, thus reducing experimental error at this stage to a minimum.

The position at which the material ceases to flow was found to be ascertainable either by observing the material from the top of the container, or by observing the termination of the discharging material at the level of the orifice. Both methods were found to be equally accurate, but the latter procedure was adopted in order to reduce the amount of fine airborne dust inhaled, especially prevalent in the crushed rock aggregates.

Flow time measurements were found to be highly reproducible in single sized material and satisfactorily reproducible in graded material, provided that the material was thoroughly mixed prior to each determination and loaded carefully into the apparatus to minimise segregation.

Compaction of the collected material was effected by the application of 25 blows from a 1 lb. "hammer" applied randomly about the exterior surface of the container. Loose material was then added to produce a dome of material above the surface and the material

97.
Excess material in the receiving container is struck off with a straight edge, and the weight of the container and loose material recorded. The material is then compacted in a standard manner, and the weight of compacted material and container ascertained.

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Flow time measurements were found to be highly reproducible in single sized material and satisfactorily reproducible in graded material, provided that the material was thoroughly mixed prior to each determination and loaded carefully into the apparatus to minimise segregation.

Compaction of the collected material was effected by the application of 25 blows from a 1½ lb. hammer applied randomly about the exterior surface of the container. Loose material was then added to produce a dome of material above the surface and the material
in the container subjected to a further 25 blows of the hammer in
the above manner. Excess material was then struck off and the
weight of the container and compacted material determined.

Due to the possibility of segregation, care was taken to
ensure that only a representative sample of material was used for
topping up.

Loose and compacted densities were found to be reproducible
to within 1%.

d. Application of efflux test results

Figures 1 and 2 show that the efflux rates of single sized
materials vary considerably with the nature of the aggregate, the
time for 100 cc. of material to flow increasing, and the volume rate
of flow decreasing, with an increasing degree of particle interaction.
The ratio of these rates to that of Ballotini produces "interaction
factors" ranging from 1 to some 1.8 for the most angular crushed rock
aggregate, providing a simple, yet informative measure of degree of
particle interaction.

Figure 3 shows that for any material, the rate of discharge
is independent of the head of material in the apparatus and the manner
in which it is filled.

Tests on 1-inch diameter steel ball bearings in appendix 2
support the assumption that the criterion of measurement is by solid
volume rate of flow, based on the density of the individual particles.

The efflux test is therefore a reliable and simple method of
measuring the degree of particle interaction of both single sized and
graded aggregates.
EFFLUX TIME
1 sec/100 cc

FIG 1

EFFLUX TIMES FOR 5/8 INCH DIAMETER ORIFICE
FIG 2

EFFLUX RATES FOR 1/8 INCH DIAMETER ORIFICE
**EFFECT OF QUANTITY OF MATERIAL AND METHOD OF FILLING ON EFFLUX TIMES**

Method of filling apparatus:

a) Material poured from top of apparatus
b) Material introduced by scoop
c) As (a) and rapped to achieve higher degree of compaction
d) As (a), test performed with apparatus inclined at 45°
ANALYSIS OF THE RATES OF DISCHARGE OF GRANULAR MATERIALS USING THE EFFLUX TEST APPARATUS

In the interests of clarity and continuity, both the experimental procedure and discussion of results will be dealt with concurrently in this section.

In order to analyse the rates of discharge of materials from the apparatus, Ballotini, nominally spherical glass particles, have been used as a datum, and have been taken in the first instance as being fully spherical.

The angles of repose of various sizes of Ballotini and sands used later in this section have been measured by means of the apparatus shown in plate 6. The values of the angles of repose have been taken as a measure of the effective coefficients of friction of the appropriate sands. Values for the full range of sand sizes of Ballotini, Thames Valley crushed flint gravel and Mountsorrel granodiorite, are shown in figure 4.

The voids in the compacted material, collected in the container after discharge from the efflux test apparatus, have been taken as a measure of the shape of the aggregate. The validity, or otherwise, of this measure will be discussed later in this thesis.

The dimensional analysis set out in appendix 3 shows that, with the elimination of extraneous variables, the equation for the rate of discharge of single sized granular material from a circular orifice of constant cone angle is in the form:
Plate 6

APPARATUS USED TO MEASURE ANGLE OF REPose
OF SINGLE-SIZED OR GRADED SANDS
COEFFICIENT OF FRICTION

Mountsorrel

Thames Valley

Ballotini

ANGLES OF FRICTION OF SINGLE SIZED MATERIALS - ANGLES OF REPOSE
\[
\frac{V}{D^2 g} = \varphi \left( \frac{D}{d} - 3 \right), \left( \mu \right), \alpha \quad \ldots \ldots 1
\]

where

\( V \) = solid volume rate of discharge

\( g \) = acceleration due to gravity

\( D \) = diameter of orifice

\( d \) = diameter of particles

\( \mu \) = coefficient of friction

\( \varphi \) = shape factor.

By using Ballotini, the shape factor \( \varphi \) is reduced to unity, leaving the volume rate of discharge \( V \) dependent upon the diameters of the orifice and particles \( (D \& d) \), and the coefficient of friction of the particles \( \mu \).

The efflux rates of nine size grades of Ballotini have been determined from the three orifice sizes, shown in figure 5, and the functions \( \frac{V}{D^2 g} \) and \( (\frac{D}{d} - 3) \) for each of the three orifices plotted on logarithmic scales.

These three plots are of a similar form and an examination of the plot for the \( \frac{3}{8} \)-inch diameter orifice, figure 6, shows that the curve falls into two distinct parts: a linear rising branch, and an curved descending branch.

The angles of repose of various Ballotini sizes will be seen from figure 4 to be constant for particles down to \( 0.5 \) mm. in diameter. This suggests a constant coefficient of friction and corresponds to the particle size range of the linear branch of figure 6. The descending branch of figure 6 would therefore appear to be due to the increasing coefficient of friction of particles sizes less than \( 0.5 \) mm. Thus, the linear portion of figure 6 represents the variation of volume rate of discharge with particle size, over the range of particle sizes where the coefficient of friction is constant, and may be expressed in the form:
FIG 5

EFFLUX RATES FOR BALLOTINI
\[ \frac{V}{D^2g^2} = K \left( \frac{D}{d} - 3 \right)^n \]

where \( K \) and \( n \) are constants depending upon the size of the orifice. The descending branch is dependent upon the increasing coefficients of friction, and may be written as:

\[ \frac{V}{D^2g^2} = K \left( \frac{D}{d} - 3 \right)^n \cdot f(\alpha) \]

where \( f(\alpha) \) is a function of the coefficient of friction \( (\alpha) \).

Now if the deviation of the descending branch of the curve from the projection of the linear portion is measured \( (\log X) \), then:

\[ \log \frac{V}{D^2g^2} = \log K \left( \frac{D}{d} - 3 \right)^n - \log X \]

or:

\[ \frac{V}{D^2g^2} = K \left( \frac{D}{d} - 3 \right)^n \cdot \frac{1}{X} \]

Comparing this equation (4) with equation 3, it is apparent that \( \frac{1}{X} = f(\alpha) \). Therefore, by plotting \( \log \frac{1}{X} \) to \( \log \left( \frac{D}{d} - 3 \right) \), the relationship \( \frac{1}{X} = f(\alpha) \) may be established. This may be substituted into equation 4, producing the relationship between volume rate of discharge, orifice and particle size:

\[ \frac{V}{D^2g^2} = K \left( \frac{D}{d} - 3 \right)^n \cdot \frac{0.125}{\alpha} \]

A summary of the plots of \( \log \frac{V}{D^2g^2} \) to \( \log \left( \frac{D}{d} - 3 \right) \) for each of the three orifices is shown in figure 7. It will be seen that for smaller values of \( \log \left( \frac{D}{d} - 3 \right) \), i.e. the larger values of particle size \( (d) \), the volume rate of discharge is greater for the 1-inch diameter orifice than for the 2-inch diameter orifice. This may be due to either one or a combined effect of two factors. Firstly,
it may be due to the edge effects of the orifice, most dominant in the smallest orifice size. Secondly, it may be due to the fact that, for a constant value of \( \frac{D}{d} - 3 \) or ratio \( \frac{D}{d} \), the value of the particle diameter \( d \) will be smaller for the \( \frac{5}{16} \)-inch orifice than for the \( \frac{3}{8} \)-inch diameter orifice. Any surface irregularities not shown up in the angle of repose may nevertheless be in action, and would be expected to be more dominant with the decreasing particle size and rapidly increasing surface area of the material, thus reducing the rates of flow.

The forms of the linear branches of the three curves in figure 7 were found to be of the form:

for the \( \frac{1}{4} \)-inch orifice : \[
\frac{V}{D^{1.25}g^{0.295}} = 0.145 \left( \frac{D}{d} - 3 \right) \]

for the \( \frac{5}{16} \)-inch orifice : \[
\frac{V}{D^{1.25}g^{0.25}} = 0.165 \left( \frac{D}{d} - 3 \right) \]

for the \( \frac{3}{8} \)-inch orifice : \[
\frac{V}{D^{1.25}g^{0.225}} = 0.180 \left( \frac{D}{d} - 3 \right) \]

Kose and Tanaka\(^{74}\) have reported the values of the indices "n" (see eqn. 2) as being between 0.27 and 0.35 for orifice diameters in the range 9 to 3.6 mm. The value of 0.295 for the \( \frac{1}{4} \)-inch diameter orifice (measured 9.25 mm.) would appear to be in accordance with their results, as would the decreasing indices with increasing orifice diameters.

The values of "n" from equation 4 were computed from the equations of the linear portions of the appropriate curves, and may be seen from figure 8 to bear good correlation to the coefficient of friction (\( \mu \)).

The constants "k" and "n" (see eqn. 2) in equations 6, 7 and 8 have been found to be related to the orifice diameter (D) in the
\[ K = 0.028 \cdot D^{0.225} \]
\[ n = 0.572 \cdot D^{-0.30} \]

As the aim of this section is to analyse the rate of discharge of sands used later in this thesis, further calculations will be confined to the \( \frac{5}{8} \)-inch diameter orifice for which the rate of discharge of spherical particles (\( V \)) is given by:
\[
\frac{V}{D^{2.5} g^2} = 0.165 \left( \frac{D}{d} - 3 \right)^{0.25} \cdot 0.33
\]
(from equations 5 and 7)
or:
\[
V = 2.02 \left( \frac{D}{d} - 3 \right)^{0.25} \cdot \frac{1}{n^{2.3}} \]

where \( V \) is expressed as the volume rate of discharge in \( \text{cm}^3/\text{sec} \).

Shape

In order to assign a numerical value to the shape factor \( C_s \) in equation 1, it was considered that a more realistic result will be obtained by an assessment of shape in bulk, rather than by the examination of a large number of individual particles. Of the methods reviewed previously, Shergold's method of assessing the angularity of an aggregate in bulk by the proportion of voids in the compacted material has been adopted. However, a different and more reproducible method of compaction has been employed as described in the previous section.

To establish the form of the function \( C_s \), two sets of tests were performed, the first using single sized aggregates, and the second using graded aggregates. This facilitated the effect of shape on the efflux rates of both single sized and graded materials.
to be investigated.

1) **Single sized aggregate** - The packing characteristics of a range of fine sands of the 18 - 25 size fraction were determined and the results shown in figures 9a and 9b. Ballotini, being of constant shape, possesses loose and compacted void ratios similar for all sizes, and the average values of these are also shown.

It will be seen from figure 9b that a linear relationship exists between the differences in loose and compacted voids, and the efflux rate. It has been assumed that truly spherical and frictionless single sized spheres would pack into their most stable position after efflux, and that the void content would not decrease under externally applied compactive forces. The efflux rate at which the difference between the loose and compacted voids is zero may therefore be used to predict the void content at which both loose and compacted voids are equal, hence determining the minimum void content attainable in this form of test. This minimum value, found to be 33%, has been used as a datum from which to calculate the shape factors of the various single sized sands in the form:

\[ Z = \frac{\text{compacted voids} - 33}{\text{loose voids}} \]

The minimum void content of 33% has not been reduced to 52% in order to ascribe a shape factor of unity to perfect frictionless spheres, since the angle of repose of these spheres would be expected to be zero. A shape factor of zero for perfect spheres, in this case, renders the form of equation 1 inapplicable for spheres, rather than producing an infinite value for the volume rate of discharge \( V \) if a shape factor of unity was employed.

It is interesting to observe that this minimum void content of 33% is the same as that defined by Sherpold.

117.
DIFFERENCE IN AGGREGATE VOIDS - %

PACING CHARACTERISTICS
OF SINGLE SIZED SANDS

Loose Voids

Compacted Voids

Fig 9b
A logarithmic plot of the six values of shape (2), plotted to their appropriate values of function of $2$ shown in figure 10, produces the final relationship for the rate of discharge of single sized particles from the $\frac{1}{4}$-inch diameter orifice:

$$ V = 0.8 \left( \frac{D}{d} - 3 \right)^{0.25} \left( \frac{C - 33}{W' \cdot 3} \right)^{0.65} \cdots$$

where $C$ = voids in compacted aggregate.

117 Graded aggregate - A procedure for determining the equivalent particle size of a graded aggregate has been developed in appendix 4. It is also shown that the efflux rate of a graded aggregate from a single petrological source, may be determined from the efflux rate of the equivalent particle size.

Appendix 5 shows that it is also possible to predict the efflux rate of a single sized aggregate combination of different sands from the proportion, and efflux rates, of the individual constituent materials.

Appendices 4 and 5 may therefore be logically extended to predict the efflux rate of any graded material, knowing the efflux rates and proportions of the various constituents.

Tests have been performed on 13 sands screened to single sizes and recombined to the aggregate grading used in the sand asphalt tests of phase 6, and shown in figure 50. The packing characteristics are shown in figures 11a and 11b.

By a similar process to that followed for the single sized aggregate tests, a minimum void content of 24% produces a shape factor ($Z$) for aggregates blended to this specific grading of:

$$ Z = \frac{\text{voids}}{24}.$$
DETERMINATION OF FUNCTION OF SHAPE
FOR SINGLE SIZED SANDS
AGGREGATE VOIDS - %

DIFFERENCE IN AGGREGATE VOIDS - %

EFLUX TIME - sec/100cc

EFLUX RATE - cc/sec

PACKING CHARACTERISTICS
OF GRADING: AGGREGATES
FIG. 12

DETERMINATION OF FUNCTION OF SHAPE
FOR GRADED AGGREGATES
The logarithmic relationship of figure 12 produces the final equation for the rate of discharge (V):

\[ V = 4.13 \frac{(C - 24)^{0.65}}{n^{2.5}} \]  

..... 11

For comparison, this equation may be written as:

\[ V = 1.39 \left( \frac{D}{d} - 3 \right)^{0.25} \frac{(C - 24)^{0.65}}{n^{2.5}} \]  

..... 12

although it is stressed that this equation is applicable only to the value of "d" appropriate to the equivalent particle diameter of the aggregate grading.

Discussion of equations 10 and 12

\[ V = 0.8 \left( \frac{D}{d} - 3 \right)^{0.25} \frac{(C - 24)^{0.65}}{n^{2.5}} \]  

..... 10

\[ V = 1.39 \left( \frac{D}{d} - 3 \right)^{0.25} \frac{(C - 24)^{0.65}}{n^{2.5}} \]  

..... 12

The indices of the shape factor (Z) in both equations 10 and 12 are identical at 0.65. This may be fortuitous, but the assessment of shape by compacted voids appears to be both logical and reliable and, as the datum values of minimum voids were evaluated rather than selected arbitrarily, a constant index may be accepted pending further investigations.

It will be seen from figures 9a and 11a that the voids in the compacted aggregate increase linearly with an increase in efflux time, in both cases the voids increasing by 5.5% with a unit increase in efflux time. Also that, for common aggregate types, the values of (C - 33) and (C - 24) for single sized and graded material respectively are similar. It is therefore evident that the expected increase in efflux rate with more densely graded materials is predicted by the
constant term, the equivalent particle diameter, and the angle of repose.

Although the rate of discharge is dependent upon the equivalent particle diameter \( d \) of the graded material, accounted for in the term \( \frac{D}{d} - 3 \), the fineness, or particle size distribution, of the material will also considerably affect the behaviour of the material in bulk. The fineness of the material, while determining the minimum voids in the compacted aggregate, does not affect the magnitude of the shape term \( C - \text{min. voids} \). It is also evident in the angles of repose and the constant terms of equations 10 and 11. The term \( \frac{K}{\mu^{2.5}} \) would therefore be expected to increase with an increasing density of grading, both \( "A" \) and \( \mu^{2.5} \) being functions of the fineness of the aggregate grading.

In order to establish the function of fineness \( k \) several graded materials of differing finenesses would have to be considered, but the shortage of time precluded this examination in the present work. However, as the aggregate grading chosen for the graded material approaches that of maximum density, evaluated from the expression:

\[
P = \sqrt{100 \frac{S}{N}} \quad \text{(ref. 112)}
\]

where
- \( P \) = % of aggregate passing given mesh size
- \( S \) = mesh size
- \( N \) = mesh size of smallest mesh passing all aggregate,

and shown in figure 50, it may be assumed that the maximum value of \( "K" \) would be in the order of 2.

Thus, the rate of discharge of a granular material from the \( \frac{1}{2} \)-inch diameter orifice would be given by the form:

\[
V = f(F) \left( \frac{D}{d} - 2 \right)^{0.25} (C - k)^{0.65} \frac{1}{\mu^{2.5}} 
\]

\[
\ldots \ldots 13
\]
where

\[ V = \text{volume rate of discharge in cc/sec.} \]
\[ f(F) = \text{function of fineness} \]
\[ D = \text{diameter of orifice} \]
\[ d = \text{diameter of equivalent particle size} \]
\[ C = \text{voids in compacted material - \%} \]
\[ M = \text{minimum voids for grading (that of perfect sphere)} \]
\[ \mu = \text{coefficient of friction, as measured by angle of repose}. \]

Tests on materials of various finenesses would define the function \( f(F) \) and lead to a theoretical determination of the minimum aggregate voids (M). The volume rate of discharge, or an interaction factor, would then be fully defined in terms of orifice and particle size, degree of aggregate interaction and aggregate grading.
FIG 13

PACKING CHARACTERISTICS OF SINGLE SIZED AND GRADED AGGREGATES (LOOSE AND COMPACTED VOIDS)
Chapter 8
Discussion of Experimental Results

Phase 1.

a. Shapes of Marshall Test Curves

In order to establish the shapes of the Marshall test curves, a total of 116 specimens of stone filled asphalt were produced. The aggregate used was Mountsorrel crushed granite throughout, with a filler content of 7%. Experimental results are shown plotted in figures 14, 15a, 16, 17a and 18.

i) Stability, figure 14 - The best smooth curve has been drawn through the average points of the experimental results. The curve shows an initial minor peak at a binder content of approximately 5%, this being 1.2% lower than that at maximum stability. The large number of specimens tested, their proximity to a regular curve, and results of previous work on similar compositions indicate that the initial minor peak is an experimental phenomenon and is not due to any experimental error. With increasing binder content the magnitude of Marshall stability rises smoothly to the initial minor peak, then rises further to the major peak of maximum stability, and finally decreases regularly.

ii) Flow, figure 15a - With increasing binder content the flow, or movement undergone in reaching the maximum load bearing capacity of the specimen, will be seen to fall very slightly to the binder content corresponding to that of the initial peak of stability, and then increase relatively rapidly. There is some evidence, supported by subsequent results, that at the high binder contents approaching that at which the mix becomes excessively fluid, the flow values
FLOW CURVES FOR STONE FILLED ASPHALTS
FIG. 15

SPECIFIC GRAVITY CURVES FOR STONE FILLED ASPHALTS.
VOIDS IN MIX

FIG 17a

FIG 17b

VOIDS IN MIXES OF STONE FILLED ASPHALTS

M/S Sand

TV Sand

M/S Stone

TV Stone

VOIDS IN MIX

(\%)

BINDER CONTENT

BINDER CONTENT

127
AGGREGATE VOIDS FILLED WITH BINDER - STONE FILLED ASPHALTS

FIG. 18

AGGREGATE VOIDS FILLED WITH BINDER - STONE FILLED ASPHALTS
tend to level off.

iii) Specific gravity, figure 16 - Over the range of binder contents below that of maximum stability, the specific gravity increases linearly with increase in binder content. At this point, with increasing binder content the specific gravity increases less rapidly to a maximum value at a binder content some 1 - 1.5% in excess of that of maximum stability and then decreases. Further experimental results suggest that this decrease resorts, ultimately, to a linear form.

iv) Voids in mix, figure 17a - The air voids in the specimens decrease approximately linearly with an increase in binder content to roughly that of the initial peak of stability, and then gradually level off to a minimum value. This minimum void content becomes apparent when the specific gravity decreases linearly.

v) Aggregate voids filled with binder, figure 18 - The proportion of the voids in the stone, sand and filler filled with binder increases with increase in binder content, levelling off to a maximum value at the higher end of the range of binder contents investigated.

b) Interpretation of the Marshall test curves

Further tests on stone filled asphalts of 50% stone content and 25% filler were performed with Thames Valley irregular flint gravel, and also on combinations of this flint gravel with the Mountsorrel crushed granite. Marshall test results for these asphalts are also included in figures 14-18.

i) Stability, figure 14 - Initial stability peaks are evident in
the stability curves of the granite and gravel combinations, but in
the case of the gravel aggregate asphalts a smooth curve has been
drawn as a possible initial peak is less well defined.

The stability at the initial minor peak of the gravel stone-
granite sand curve is of a lower magnitude than the stability at the
major peak, as is the curve of the granite aggregate asphalts.
However, the stability at the initial peak of the granite stone-
gravel sand curve appears to be in excess of the second major peak
value. This phenomenon, together with the fact that the magnitude
of the initial peaks of the curves of similar stone type are
approximately equal, suggests that the initial peak is due to some
initial load bearing characteristic of the coarse aggregate skeleton.
With an increase of binder content in the order of 1%, the major
stability peak is achieved and is dependent in its magnitude and
binder content upon the type of stone and sand.

The stability curve of the gravel asphalt indicates a relatively
low stability value of 1600 lb. This is probably due to the poor
degree of particle interaction of the smooth textured aggregate
resulting in a low angle of internal friction. The main contribution
to the stability would therefore appear to be from the filler-binder
matrix, rather than from the aggregate particle interaction.

The main peak of the gravel stone - granite sand asphalt
stability curve will be seen to be only 120 lb. higher than that of
the gravel asphalt, and occurring at a binder content 0.7% higher at
5.7%. This increase in binder content is accounted for by the
increase in the surface area to be covered by the binder, when
replacing the relatively rounded gravel of the gravel asphalt by the
more angular crushed rock fines. The small increase in stability
is due to the higher degree of particle interlock of the crushed
rock fines. The relatively small stability increase may be accounted
for by the interference in the interaction of the angular fines by the smooth textured more rounded coarse aggregate.

By replacing the gravel stone in the gravel stone - granite sand asphalt with the crushed rock stone, a marked increase in stability is achieved, increasing by 860 lb. to 2380 lb. It is apparent that with the granite aggregate throughout, the rough surface texture and the angular nature of the aggregate combine to their best advantage to produce a high stability mix. The binder content at the main peak of stability also increases by 0.5%, again accounted for by the increasing surface area of the coarse aggregate.

A similar explanation may be put forward to illustrate the increase in stability and binder content by transition from the gravel aggregate asphalt to the granite aggregate asphalt by way of altering the type of stone in the first instance, rather than the sand as followed above:

The replacement of the gravel stone of the gravel asphalt with the crushed granite increases the stability by 500 lb. and the binder content by 0.5%. The substitution of the granite fines, resulting in an asphalt of granite throughout, further increases the stability by 480 lb. to 2580 lb., and the binder content by 0.7% to 6.2%

The postulate that the change in the binder content at optimum with alteration of either the stone or sand type is due almost entirely to the different surface area of the aggregate, is validated by the experimental data. Results show that the substitution of crushed granite for gravel fines requires an increase in binder content of 0.7%, irrespective of the type of stone present. Further that the substitution of crushed granite for gravel stone requires an increase in binder content of 0.5%, irrespective of the type of sand present. A certain proportion of the additional binder may,
however, be required to accommodate the increasing voids, and this aspect will be considered in more detail later in this chapter.

The effect on stability at the major peak upon altering the type of either stone or sand is not independent of the type of the remaining constituent, indicating that the magnitude of the main peak of stability is a combined effect of both stone and sand type.

It is therefore suggested that the shape of the Marshall stability curve of a continuously graded stone filled asphalt may be accounted for as follows:

From very low binder contents, stability increases gradually with increasing binder contents. This increasing strength is due to the additional binder providing a greater bond to the aggregate particles, thus increasing, to some extent, the degree of friction between the aggregate particles. When there is sufficient binder to bind the aggregate particles together, enabling the aggregate to produce its maximum potential, or inherent stability, the initial minor peak is achieved. Throughout this stage the movement undergone by the specimen under test is relatively constant, falling only slightly due to the increasing binder content providing a greater contribution to the stability of the mix. This supports the assumption that the initial aggregate skeleton provides the initial stability and that the binder serves mainly to restrain the dry aggregate rather than to bind the particles together. The use of a different viscosity binder, or a series of tests at a different testing temperature would be necessary to investigate this theory further.

Upon further increasing the binder content above that of the initial minor peak, the surplus binder acts as a lubricant to the aggregate particles. The specimen will compress under loading by increasing amounts as the binder content is increased and this is
illustrated by the rising flow values at this stage. If the basic aggregate skeleton is sufficiently rigid to retain its initial or inherent stability, then the stability value will remain constant. Conversely, if the basic aggregate skeleton is inherently unstable, then the stability will be reduced with the lubricating action of the excess binder.

It will be seen from the stability curves in figure 14 that the two aggregate compositions containing the crushed granite fines retain their initial inherent stability while the stability of the composition with the flint gravel fines is reduced after the initial minor peak. This suggests that the lubricating effect of the excess binder is increased by the presence of a rounded sand, and reduced by a more angular rough-textured sand. The sand fraction, therefore, while apparently not contributing to any great extent to the magnitude of the initial inherent stability, does have a considerable effect on the behaviour of the material just after this peak.

As the binder content is further increased, the deformation of the specimen under loading also increases. This indicates that the aggregate particles undergo a form of secondary compaction in which the particles are bedded down into a more stable position. The particles become more closely associated and the resulting increase in particle interaction produces higher bearing capacities of the specimens and hence higher stabilities.

Stability then increases to a point at which there is just sufficient binder to permit the aggregate particles to compress into their most stable position under the action of the applied loading and the major stability peak is achieved.

This secondary state of compaction may result in stabilities in excess of the initial inherent stability, as is the case with the
granite aggregate, where the increased compaction results in a sand-filler-binder matrix binding an angular crushed rock stone. In the case of the granite stone - gravel sand mixtures, it is apparent that the increased compaction, while providing an increased stability, does not achieve a stability as high as that of the initial minor peak. This is probably due to the packing characteristics of the smooth gravel stone and is reflected in subsequent graphs.

Upon further increasing the binder content beyond the point of maximum stability, the lubricating effect of the surplus binder results in a considerable drop in stability, probably reducing ultimately to the stability of the binder.

It will be noticed that the stability curve of the gravel stone - gravel sand combination of aggregates appears to decrease in magnitude between 2\% and 3\% binder. This may suggest the presence of a stability from the dry aggregate, decreasing slightly with the introduction of significant amounts of binder, until the binder content is sufficiently large to cover the aggregate particles and contribute towards the initial minor stability peak. The stability then behaves as described previously.

The apparent absence of an initial minor peak in the stability curve of the gravel asphalt may be due to the lack of any inherent stability resulting from the smooth nature of the aggregate.

The initial minor peak of stability would appear to be dependent primarily upon the nature of the coarse aggregate, while the major peak is dependent upon the nature of the coarse and fine aggregate and also the strength of the sand-filler-binder matrix.
ii) Flow, figures 15a and 15b - Flow curves for all four aggregate combinations follow a similar pattern, decreasing slightly with increasing binder content to a binder content in the region of that of the initial minor peak of stability, and then increasing over the remaining range of binder contents.

Results will be seen to form smooth curves with relatively little deviation from the lines drawn. In the majority of cases the deviation is within the limits of experimental error.

Flow is a measure of the movement undergone by the specimen under loading and it is evident that asphalts composed of the more angular aggregates experience a greater movement in achieving their final degrees of compaction. This is due to the aggregate's initial resistance to impact compaction during the preparation of the specimen.

The flow values of the more angular sand are apparently more tolerant to changes in binder content, especially at the higher binder contents due probably to the high degree of particle interaction and hence the lesser degree of dependence upon the binder quantity.

The sand type is evidently the major factor influencing the flow properties of the stone filled asphalts investigated. This is especially noticeable at binder contents in excess of those of maximum stability where the type of stone has apparently no effect upon the flow values. This supports the previous suggestion that the stone type has little effect on the behaviour of continuously graded stone filled asphalts in the higher binder contents, and that the sand is a prominent factor throughout.

These findings are in accordance with those of Lees, Stanton and Hveem, Campen and Smith, and others.

iii) Specific gravity, figure 16 - As the binder content increases,
the specific gravity of the untested specimens of stone filled asphalt increases linearly at a rate of 0.04 per unit increase in binder content. At the binder content of maximum stability, the rate of increase becomes less rapid until a maximum value is achieved and finally decreases.

Experimental points are subject to errors of up to 1%, and occur mainly in the region of the lower binder contents where reproducible compaction is more difficult.

The specific gravity of the compacted specimen would be expected to be dependent upon the specimen height when compacted in a standard manner. Figure 19 shows that for the crushed granite asphalts the specific gravity of the specimens of 3% binder decreases conspicuously with an increase in specimen height. A similar trend is evident in the specimens of 7% binder, although less acute. The gravel asphalts, however, are less susceptible to changes in specimen height, and hence provide less variation in specific gravity determinations.

These changes in specific gravity are attributed to the changing angles of internal friction of the mixtures under compaction. The rough textured granite with low binder content transmits the compactive effort less readily than does the smooth textured gravel which transmits the effort to the centre of the specimen and compacts more thoroughly.

Over the linear portion, the increase in specific gravity with increasing binder content is due partly to the reduction in aggregate voids, and partly to the lubricating effect of the binder providing a greater degree of compaction of the specimen.

An increasing binder content results in an increasing specific gravity until the binder content is such that all the aggregate particles are coated with a film of binder, are compacted into their
FIG. 19

VARIATION OF SPECIFIC GRAVITY OF SPECIMEN WITH HEIGHT

FOR STONE FILLED ASPHALTS
most stable position and a large proportion of the voids in the dry aggregate are filled with binder. At this point additional binder tends to hold apart the aggregate particles causing a steady reduction in specific gravity and stability.

The curve of the gravel stone - granite sand combination will be seen to level off at the lower end. This may well be due to the bulking effect of the binder at very low contents where the addition of small amounts of binder would tend to hold apart the aggregate particles. This bulking effect would continue until the binder is in such quantity that it permits a closer packing of the particles during compaction.

The slope of the linear portion of the specific gravity curve would be expected to be dependent mainly upon the particle size distribution of the aggregate which determines, to a large extent, the quantity of voids in the dry aggregate. Also, but to a lesser extent, upon the degree of interference of the aggregate particles.

iv) Voids in mix, figures 17a and 17b - With increasing binder content the air voids in the compacted specimen prior to testing are reduced almost linearly. Similar to the specific gravities, this is attributed partly to the binder part-filling the aggregate voids and partly to the increasing binder permitting a higher degree of compaction. When the binder content is such that the air voids are filled as far as possible, excess binder serves only to hold apart the aggregate particles, and produces no further reduction in voids. The minimum void content may be due to air pockets in the surface of the aggregate particles into which the binder cannot penetrate, the semi-porous nature of the filler, and accumulative errors in specific gravity determinations of both the aggregate and the specimen.

The voids in the granite sand asphalts will be seen to fall
slowly at first then assume a more rapid linear relationship. This is probably due to the higher degree of particle interaction of the granite fines resisting compaction until the binder content is in sufficient quantity to permit movement. Initial void reduction is due mainly to the added binder part filling the air voids with only slight compaction of the aggregate particles.

Both linear portions of the granite and gravel sands are of equal slope, decreasing by approximately $3\%$ voids per unit increase in binder content.

As was the case with Marshall flow, the type of sand influences to a considerable extent the quantity of air voids in the compacted specimen prior to load testing. The type of stone appears to influence only the final limiting value of the voids, the gravel stone entraining more air voids due to the difficulty of packing around its smooth semi-rounded surface.

v) Voids filled with binder, figure 18 - The proportion of voids in the dry aggregate which are filled with binder increase with increasing binder content in a manner similar to that in which the voids in the compacted specimen decrease. This is linear at first and then levelling off to a constant value, governed by the combined effect of the stone and sand.

Phase 2.

Effect of aggregate grading on the behaviour of stone filled asphalt

To investigate the effect of aggregate grading on the behaviour of stone filled asphalts, four aggregate gradings were selected. Each grading had the same maximum and minimum particle size and stone contents ranged from $30\%$ to $60\%$. The filler content remained constant at $7\%$. The aggregate gradings are shown in figure 48 and
test results in figures 20-24.

i) Stability, figure 20 - The close proximity of the test results of the higher stone content asphalts lead to some confusion in the stability curves at lower binder contents. However, it is quite evident that the initial peak of stability becomes considerably less pronounced as the stone content is increased, showing a distinct absence of peak in the 60% stone content asphalt.

On the hypothesis of an initial stability from the basic aggregate skeleton, it would appear that as the stone content is reduced in the 60% to 40% stone content range, i.e. the concentration of the sand fraction is increased, the basic aggregate skeleton strength is increased. Conversely, the more dominant initial minor peak of stability in the lower stone content asphalts would indicate a less stable initial stability which is considerably reduced while the aggregate particles undergo the preliminary stages of the secondary compaction. This latter interpretation would appear to be more logical since the aggregate grading of maximum density and hence in theory, though not in practice, of maximum stability, would have a stone content of 64.5% (see references 106 and 112). This is a little higher than the maximum stone content investigated. Further, the initial inherent stability of the 30% stone content asphalt is considerably below that of the 40% and 50% stone content asphalts, supporting the latter theory.

In summary, it would appear that as the stone content is reduced, the initial inherent stability would be reduced and become less stable.

It will be seen that over the range 40% to 60% stone content, the magnitude of maximum stability is sensibly constant, supporting the results of previous work by the author"119. Also that with a reduction in stone content to 30%, the maximum stability is considerably
FIG. 20

STABILITY CURVES FOR STONE FILLED ASPHALTS
WITH VARYING STONE CONTENTS
FIG. 21

FLOW CURVES FOR STONE FILLED ASPHALTS
WITH VARYING STONE CONTENTS
FIG 22

SPECIFIC GRAVITY CURVES FOR STONE FILLED ASPHALTS
WITH VARYING STONE CONTENT
FIG. 23

VOIDS IN MIXES OF STONE FILLED ASPHALTS

WITH VARYING STONE CONTENTS
AGGREGATE VOIDS FILLED WITH BINDER - %

FIG. 24

AGGREGATE VOIDS FILLED WITH BINDER - STONE FILLED ASPHALTS

WITH VARYING STONE CONTENTS
Optimum Marshall test properties for stone filled asphalts with varying stone content.

(Continuous aggregate gradings - Granite aggregate - 7% filler.)
reduced. This is in accordance with the findings of Licker, Roscoe, and the Road Research Laboratory. It would appear to agree with the principle that more densely graded materials provide greater load bearing qualities.

Maximum stability have been plotted against the in-place asphalt content in figure 36, and it will be seen that with increasing asphalt content the maximum stability rises to a peak at a stone of about 2%, then declines. The rise is due to the increasing quality of the aggregate grading, and the decline, but not a theoretically best asphalt content at maximum aggregate stability of 7% stone, due to the increasing degree of separability caused by the binders required at maximum stability. The binder content is a result of the relatively low asphalt and void content of the aggregate.

The drift in binder content at maximum stability with increasing grading, in figure 30, is accounted for by the chance in both aggregate voids to be filled with binder and the surface area of the aggregate, with a changing proportion of fine aggregate. The decline in binder content at maximum stability with increasing stone content is illustrated graphically in figure 37.

ii) Flow, figure 31. - The flow value at the binder content of maximum stability is approximately the same for the three asphalt content asphalts in the 40% to 60% range. This indicates, suggested previously, that the stone content of asphalt in this range has little effect on the Marshall test properties at stability. The flow of the 30% stone content asphalt is considerably less than those of the 40% - 60% range at maximum stability, indicating a more thorough degree of compaction in the preparation of the specimen resulting in less movement under test.
The basic flow value of the 60% stone content asphalt does not reduce to the same value as that of the lower stone content asphalts. This may be attributed to the difficulty in compacting such a densely graded material, especially at such low binder contents.

iii) Specific gravity, figure 22 - As the aggregate densities are equal for all four aggregate gradings, it is possible to compare the packing characteristics of the various asphalts from the specific gravity curves.

As stated previously, the particle size distribution of the aggregate would be expected to influence the rate of increase of specific gravity with increasing binder content over the linear portion of the curve. This is evident in figure 22 where the rate of increase will be seen to be less as the stone content is reduced. As the stone content decreases, the proportion of fine aggregate increases thus producing a greater surface area to be coated with binder, and a greater volume of voids to be filled.

The maximum value of specific gravity will be seen to increase with the stone content over the range 30% to 50% but decrease slightly at a stone content of 60%. The increase is due to the increasing density of the graded aggregate, and the decrease to the drop in workability. These maximum values are presented with stone content in figure 26, and a maximum specific gravity is predicted at a stone content of 52.5%.

The binder content at maximum specific gravity reduces with an increasing stone content, as shown in figure 28, in a similar manner to that of maximum stability, and for the same reasons.

iv) Voids in mix, figure 23 - As would be expected, the void curves of the stone filled asphalts are of a similar shape, and with
A drift in binder content appropriate to the aggregate grading. The minimum void contents are evidently governed by the nature of the aggregate grading, being dependent, as before, upon the aggregate packing and workability.

The Air Ministry Specification No. 201 in its recommendation for paved areas of Marshall asphalt stipulates in schedule V a void content of between 3% and 5%. The binder content at 4% voids has therefore been shown against the corresponding stone content in figure 29, and for the mix requiring the minimum amount of binder, a stone content of 52.5% would appear appropriate.

v) Aggregate voids filled with binder, figure 34 - As was the case with the specific gravity curves, the initial slopes of the curves of voids in dry aggregate filled with binder are related to the compacted voids in the dry aggregate, and the total surface area of the aggregate particles. The increase with increasing binder content is less acute with the lower stone content asphalts where higher initial aggregate voids and surface area require a greater amount of binder.

The Air Ministry calls for between 75% and 82% aggregate voids filled with binder, and the binder content at 78% voids filled with binder has been plotted to stone content in figure 30. It is interesting to observe that the stone content requiring the least binder for this condition is again 52.5%, suggesting this figure as the ideal stone content for the particular aggregate type, filler content and binder used in this investigation.

This ideal figure of 52.5% stone content is that at which the mixture would be at its densest and most highly stable form, requiring the minimum amount of binder for all conditions.

The Air Ministry range of stone content for Marshall asphalt is 40% to 52.5% and it is interesting to observe that the upper limit
specified in that of maximum density as determined in the course of
the present work. The lower limit of 40% is that below which
stability drops considerably, and void conditions fall outside the
specified ranges. The range 40% to 52.5% has been found experiment-
ally to be that at which mixes are at their most favourable conditions
and are relatively insensitive to small changes in binder content.

Phase 2.

Comparison of gap- and continuously-graded asphalts.

Experimental results discussed earlier in this thesis indicate
that the potential strength of continuously graded asphalt mixtures is
dependent mainly upon the degree of interlock of the aggregate particles
and partly upon the strength of the sand-filler-binder matrix.

Hot rolled asphalts employed in this country are mainly of the
gap graded type as specified in B.S. 594. By nature of their
aggregate grading these mixes rely to a considerable extent upon the
strength of the sand-filler-binder matrix for their ultimate stability.

In order to investigate the different characteristics of dense
asphalt mixtures, the test results of the continuously graded granite
stone - gravel sand asphalts described in phase 1, have been compared
with those of a gap graded asphalt. This gap graded asphalt has a
continuously graded stone and sand, deficient in the 7-25 sieve size
sand fraction and the aggregate grading is shown in figure 45. A
further single sized stone asphalt, with an "asphalt sand" mainly of
the 52-100 sieve size range, has been investigated, but the inconsist-
ency in results of specific gravity determinations precludes a study
of the specific gravity and void characteristics. Marshall test
results are, however, discussed in context.

To avoid confusion, the gap graded asphalt with the continuously
graded stone will be referred to as the gap graded asphalt, and the
Second gap graded asphalt with the single sized stone will be suitably distinguished when the occasion arises.

1) Stability, figure 31 - It will be readily observed that the major peaks of stability for the continuously graded and gap graded asphalts are similar at approximately 2100 lb. This supports the premise that the main stability is a combined effect of the interaction and rigidity of the coarse aggregate skeleton and the strength of the sand-filler-binder matrix. It is apparent that the larger sand size fraction at this point serves solely to part fill the voids left in the stone skeleton and, where absent, the resulting voids are filled by the binding matrix. This would account for the additional 1% of binder required for maximum stability of the gap graded asphalt.

The initial minor peak of stability, accounted for previously by the strength of the aggregate skeleton, is less in the case of the gap graded asphalt probably due to the absence of the larger sand sizes. The latter asphalt is, however, more stable, as illustrated by the apparent lack of sensitivity of stability to small changes in binder content, and by the constant flow values over this range of binder contents.

Stability results of the gap graded asphalt with the single sized stone are somewhat erratic, but the suggested shape of the stability curve is depicted in figure 31. With the exception of the higher stability at a binder content of 5%, all stabilities are of sensibly equal magnitude at 1240 lb. over the limited range of binder contents investigated. This is most probably accounted for by the strength of the sand-filler-binder matrix, the stone serving purely to reduce the volume of the matrix. As the stone content is evidently insufficient to form a skeleton, it is probable that the higher stabilities produced at 5% binder content are due to experimental error in production.
STABILITY CURVES FOR GAP AND CONTINUOUSLY GRADED AGGREGATES IN STONE FILLED ASPHALTS
FLOW CURVES FOR GAP AND CONTINUOUSLY GRADED AGGREGATES IN STONE FILLED ASPHALTS
AGGREGATE VOIDS FILLED WITH BINDER — STONE FILLED ASPHALTS

WITH GAP AND CONTINUOUSLY- GRADED AGGREGATES
However, the three curves do establish satisfactorily that the magnitude of maximum stability is dependent upon the rigidity of the coarse aggregate skeleton, where present, and upon the strength of the sand-filler-binder matrix.

ii) Flow, figure 32 - The flow curves of the continuously graded and gap graded asphalts are of a similar form. A considerable drift in binder content is evident, resulting in less variation in flow for the gap graded material than for the continuously graded material in the binder contents close to that of maximum stability.

The flow curve of the gap graded asphalt of single sized stone appears to increase almost linearly, accounted for by the increasing quantity of the binder present, and hence the greater fluidity of the mix.

iii) - v) Specific gravity and dependent variables, figures 32-33. - Apart from the expected drift in binder content, the gap graded asphalt appears to have a higher proportion of voids accounted for by the greater degree of voids in the compacted dry aggregate.

The curves are of the predicted form, and in the light of previous discussions, are self explanatory.

In summary, the stabilities of the continuously graded and gap graded asphalts are of a similar magnitude, but the gap graded asphalt accommodates a larger amount of binder and is less susceptible to small changes in binder content. At optimum binder content, although greater, the flow values of the gap graded asphalt are less variable than those of the continuously graded asphalt.

It would appear, therefore, that at the conditions examined, the gap graded asphalt is the more stable mixture accommodating a
larger amount of binder, but flow values are greater and they possess a higher proportion of air voids.

Phase 4.

The efflux test

Detailed results and the discussion of experimental work been dealt with concurrently in chapter 6. Although the lack of time precluded a complete analysis of the efflux test, results indicate quite clearly that the measure of rate of discharge of aggregate particles from a circular orifice is a reproducible, simple and meaningful way of assessing the combined effect of particle shape, angularity and surface texture in the action of both single sized and graded aggregates.

Phase 5.

Tests on sand asphalts

A series of twelve sands was investigated, each with a filler content of 10% and blended to a common aggregate grading shown in figure 50. This aggregate grading, taken from the Road Research Laboratory's Research Note No. 3570, approaches that of maximum density, and was employed in preference to the grading of maximum density following the experimental results of the investigation into stone filled asphalts.

Aggregates investigated include nominally spherical glass particles, rounded silica sand, rounded flint gravel, irregular flint gravel, two crushed granites, crushed porphyry, basalt and limestone. Combinations of gravel, granite and porphyry were also studied.
STABILITY CURVES FOR SAND ASPHALTS
FLOW CURVES FOR SAND ASPHALTS

FIG 37
SPECIFIC GRAVITY CURVES FOR SAND ASPHALTS

St Ives (X)
Leighton Buzzard (Δ)
Thames Valley (+)
Mountsorrel (Ο)
Croby (Ο)
Limestone (Ο)
Basalt (Δ)
Harden (Ο)

Combinations (Ο)
1/2 TV 1/2 M/S (X)
1/2 Har 2/3 TV (Ο)
1/3 TV 2/3 M/S (X)

FIG 38

SPECIFIC GRAVITY

> 320
> 280
> 240
> 200
> 160
> 120
> 80
> 40

BINDER CONTENT - %
VOIDS IN MIXES OF SAND ASPHALTS

FIG 39
AGGREGATE VOIDS FILLED WITH BINDER - %

FIG. 40

AGGREGATE VOIDS FILLED WITH BINDER - SAND ASPHALTS
Test results and theoretical determinations are shown in figures 34-41.

The initial peaks of stability, seen in figure 36, are less dominant in the stability curves of the more rounded sands. This supports the theory of an initial inherent stability from the coarse aggregate structure. The Thames Valley irregular flint gravel becomes increasingly more rounded as the particle size decreases, due to attrition and the crushed nature of the larger particle sizes, and its high initial peak of stability, in asphalt form, validates the theory that the initial inherent stability is dependent mainly upon the nature of the coarse fraction of the aggregate.

It is unfortunate that the limited series of Leighton Buzzard rounded silica sand asphalts did not reach its peak of maximum stability in the range of binder contents examined. The lack of time did not permit the range to be extended.

The Harden crushed porphyry series of asphalts will be seen to produce similar values of stability over the range of binder contents examined. Although no logical explanation of this phenomenon can be offered, the conformation of test results with physical properties in subsequent relationships precludes the possibility of experimental error in the region of the predicted maximum stability.

Basic flow values of the asphalts of more angular aggregate will be seen from figure 37 to be greater than those of more rounded aggregate, due to the lesser degree of initial compaction during preparation, and consequently greater movement undergone before achieving the most stable load-bearing position.

All curves of voids in mix and aggregate voids filled with
binder (figures 39 and 40 respectively) appear to level off at similar values, the consistency of results being attributed to the identical aggregate grading of all aggregate types.

Aggregate voids in mix have also been evaluated and are shown for all sand types in figure 41. With increasing binder content, aggregate voids decrease to a minimum value and then increase slightly. The decrease in voids is attributed to the increase in workability resulting from increasing binder content, producing mixtures of greater compaction and fewer aggregate voids. At minimum voids the binder content is in sufficient quantity to permit the aggregate particles to adopt their closest form during compaction. The addition of binder at this stage serves only to hold the aggregate particles, resulting in an increasing proportion of aggregate voids.

It was apparent that the binder content at minimum aggregate voids in mix corresponded to that of maximum stability, for both stone filled and sand asphalts, shown graphically in figure 41, indicate conclusively that the binder content at maximum stability is the same as the binder content at minimum aggregate voids in mix. This illustrates that the maximum load bearing capacity of an asphalt will be achieved when the binder content is such that the aggregate particles are permitted to assume their closest form of packing during compaction in preparation and under loading at test.

The binder content of the limestone sand asphalt at maximum stability will be seen to be some 10% higher than the binder content at the minimum voids in the mixed aggregate. This is most probably due to the high proportion of fine dust present in the sieved aggregate requiring additional binder to coat it, and the resulting increase in strength of the binder matrix may account for the
Certain results have been omitted for clarity. Results of all asphalts are tabulated in the text.
FIG 42

RELATIONSHIP BETWEEN BINDER CONTENTS AT MAXIMUM STABILITY AND MINIMUM AGGREGATE VOIDS.
increased stability evident in subsequent examinations.

The binder content at minimum aggregate voids has been shown, in figure 43, to increase linearly with the proportion of minimum voids in the mixed aggregate. This signifies that the surface area characteristics of the aggregate particles are reflected in the void content of the mixed aggregate, and that aggregate packing, rather than surface area, is the dominant factor governing the appropriate amount of binder required for maximum stability.

The extrapolated linear relationship will be seen not to pass through the origin, but to suggest a small proportion of voids in the mixed aggregate at zero binder content. This small void content may well be associated with the minimum void content of mixed materials discussed previously and seen in figures 17a, 17b, 23, 34 and 39.

The premise that the movement undergone in attaining the maximum load bearing quality of a specimen is due to the bedding down, or secondary compaction, of the aggregate particles, is confirmed by figure 44. It signifies that the flow of sand asphalts at maximum stability is directly proportional to the minimum amount of aggregate voids in the mixture. As the proportions of minimum aggregate voids in the mixtures increase, the movement undergone in attaining the secondary and final state of compaction increases, resulting in higher flow values.

Variations in the linear relationship are due to experimental error, and the recognised variation in flow measurement.

A similar relationship is tentatively shown for the stone filled asphalts, where the increase in flow with increasing minimum aggregate voids will be more rapid than for the sand asphalts, due
increased stability evident in subsequent examinations.

The binder content at minimum aggregate voids has been shown, in figure 42, to increase linearly with the proportion of minimum voids in the mixed aggregate. This signifies that the surface area characteristics of the aggregate particles are reflected in the void content of the mixed aggregate, and that aggregate packing, rather than surface area, is the dominant factor governing the appropriate amount of binder required for maximum stability.

The extrapolated linear relationship will be seen not to pass through the origin, but to suggest a small proportion of voids in the mixed aggregate at zero binder content. This small void content may well be associated with the minimum void content of mixed materials discussed previously and seen in figures 17a, 17b, 23, 24 and 49.

The premise that the movement undergone in attaining the maximum load bearing quality of a specimen is due to the bedding down, or secondary compaction, of the aggregate particles, is confirmed by figure 44. It signifies that the flow of sand asphalts at maximum stability is directly proportional to the minimum amount of aggregate voids in the mixture. As the proportions of minimum aggregate voids in the mixtures increase, the movement undergone in attaining the secondary and final state of compaction increases, resulting in higher flow values.

Variations in the linear relationship are due to experimental error, and the recognised variation in flow measurement.

A similar relationship is tentatively shown for the stone filled asphalts, where the increase in flow with increasing minimum aggregate voids will be more rapid than for the sand asphalts, due
RELATIONSHIP BETWEEN BINDER CONTENT AT MINIMUM AGGREGATE VOIDS AND MINIMUM AGGREGATE VOIDS
FLOW AT MINIMUM AGGREGATE VOIDS - X 0.01

FIG. 44

RELATIONSHIP BETWEEN FLOW AT MINIMUM AGGREGATE VOIDS
AND MINIMUM AGGREGATE VOIDS.
RELATIONSHIP BETWEEN MINIMUM AGGREGATE VOIDS IN MIXES OF SAND ASPHALTS AND EFFLUX RATES OF DRY AGGREGATES.
RELATIONSHIP BETWEEN MAXIMUM STABILITIES OF SAND ASPHALTS AND EFFLUX TIMES OF DRY AGGREGATES
to a similar proportion of voids in the mixed aggregate representing a lesser degree of compaction in stone filled asphalts.

The minimum aggregate voids of the sand asphalt mixtures will be seen from figure 45 to be linearly related to the efflux rate of the dry aggregate particles. This indicates that the form of particle interaction of the aggregate in its dry state also dominates its performance in a bituminous mixture. The linear relationship in figure 45 is peculiar to the particular aggregate grading selected for this investigation. The slope of the relationship would be expected to decrease as the grading of the continuously graded aggregate tended to become finer, resorting, ultimately, to a horizontal relationship for single sized material. In this case the minimum aggregate voids would remain constant while the efflux rate increased with decreasing particle size.

Maximum Marshall stability has been plotted to the efflux time of 100 cc. of material in figure 46, and the limited number of results, with the exception of the limestone and the basalt, suggest a linear relationship. As the efflux time is related linearly to the compacted voids in the dry aggregate (see figure 14a), the stability will be linearly related to the compacted voids in the dry aggregate, again supporting the theory of stability.

The increased stability of the limestone and basalt may be due to the high dust content and its affinity to the larger aggregate particles, and/or the greater adhesive properties of these aggregate types for binder. Mathews et al.\textsuperscript{175}, Skidmore\textsuperscript{176}, Knight\textsuperscript{169} and Richards and Varma\textsuperscript{177} have reported a greater bond between a binder and limestone or basalt than with other aggregates. It may be that these hydrophobic aggregates produce a higher stability resulting
from the combined effect of particle interaction and binder adhesion, rather than the greater reliance on particle interaction of the hydrophilic, more acidic aggregates.

Figure 17 shows the relationship between the loose and compacted voids in the dry aggregate and the corresponding voids in the bituminous mixture at maximum stability and minimum aggregate voids. The minimum values of voids are derived from those of figures 11a and 45, and the relationship is confined specifically to the particular aggregate grading investigated.

The Marshall test procedure has been employed throughout the course of the investigation to study the behaviour of asphalt mixes under quasi-static loading. While not necessarily relating to practical experience, results have provided a basic understanding of the mechanical properties of both stone filled and sand asphalt mixes. Further, the use of the efflux test apparatus has provided a simple, yet reliable, method of predicting the behaviour of aggregate particles in sand asphalt mixes.

In order to predict the resistance of surfacings to deformation under heavy wheel loading, the wheel-tracking apparatus is recommended as a means of examining the performance of both stone filled and sand asphalt mixes under repeated dynamic loading. Although the wheel-tracking apparatus shown in plate 7 was set up and preliminary tests undertaken, it was not possible in the time available to include this aspect in the present investigation.
MINIMUM AGGREGATE VOIDS IN MIX - %

COMPACTED VOIDS

LOOSE VOIDS

Ballotini

Datum: see figs. 11a & 45

VOIDS IN DRY AGGREGATE - %

RELATIONSHIP BETWEEN MINIMUM AGGREGATE VOIDS IN SAND ASPHALTS AND LOOSE AND COMPACTED VOIDS IN DRY AGGREGATES

175.
Fig. 50

AGGREGATE GRADING CURVE OF SAND ASPHALTS
The experimental results have been discussed in detail in the preceding section of this thesis and the following main conclusions are drawn:

1. Variation of Marshall stability with binder content may be considered in two distinct stages. At low binder contents, the stability is dependent mainly upon the initial load bearing characteristics of the basic aggregate structure, but at higher binder contents, when the aggregate particles are further compacted during the test, the degree of particle interaction of the coarse aggregate and the strength of the sand-filler-binder matrix are each contributory factors.

2. The nature of the stone in stone filled asphalts is important in contributing to the strength of the basic aggregate structure, and to a lesser extent, the strength of the mix at the major peak of stability. However, changes in stone content within the range of 40% to 60% have little effect on the Marshall stability of continuously graded stone filled asphalts.

3. Tests on sand asphalts support the postulate that maximum stability is achieved when the quantity of binder present is just sufficient to permit the aggregate particles to compact to the maximum degree during the preparation of the samples, and subsequently to adopt their closest form of packing during the test.

4. Aggregates displaying pronounced particle interaction and hence high aggregate voids, both as a separate entity and as a
constituent material in a bituminous mix, possess potentially high load bearing capacities.

5. The flow, or movement undergone in sustaining the load, is dependent upon the initial degree of compaction of the aggregate particles, mixes of higher aggregate voids displaying larger flow values.

The flow characteristics of mixtures of continuously graded stone filled asphalt are dependent mainly upon the nature of the fine aggregate, and are influenced only in the first instance, to a small extent, by the nature of the stone. The quantity of stone, in influencing the initial degree of compaction, affects the magnitude and variation of flow especially at the higher binder contents. These findings may be logically extended by considering the coarse and fine size fractions of sand asphalts in a similar manner.

6. The rate of change in voids and aggregate voids filled with binder with increasing binder content is dependent upon the fineness of the aggregate grading. This finding applies to both stone filled and sand asphalt mixes and indicates that the packing characteristics govern the quantity of aggregate voids to be filled with binder. There appears to be a minimum proportion of air voids which are not reduced upon increasing the binder content, and this has been found to be characteristic of all the asphalts examined.

The proportion of voids and aggregate voids filled with binder in stone filled asphalts is governed almost exclusively by the nature of the sand.

7. The efflux test has proved to be a simple and reliable method of assessing the combined effect of particle angularity, shape and
texture of both single sized particles and a graded aggregate.
The test may be employed to predict the binder content of a sand asphalt required for maximum stability, the magnitude of this stability, the corresponding flow and the void characteristics of the mixed aggregate.

The diameter of the orifice required to avoid blocking restricts, for practical purposes, the use of the efflux test apparatus to the study of aggregate particles in the sand size range. However, as the sand fractions govern to a considerable extent the behaviour of stone filled asphalts and as the suitability of most sands in practice is a matter for conjecture, this physical test would appear to be applicable in its present form.

The conclusions drawn apply only to the range of materials and test conditions adopted in this investigation. They provide a basis for an understanding of the behaviour of densely graded bituminous materials under load, and it is especially useful that the test adopted for assessing the degree of particle interaction can be correlated with Marshall behaviour.

It is considered that the topic merits further investigation, especially in relation to examining the possibility of correlation with the service behaviour of asphalts, and suggestions are given in the following section regarding the aspects which would appear to merit detailed study.
CHAPTER 10

RECOMMENDATIONS FOR FURTHER WORK

The work reported in this thesis is limited to a study of the effect of aggregate type and grading on the Marshall parameters of dense asphalt mixes. The work could, with advantage, be extended in order that the following variables be studied separately:

1. Effect of filler content, especially in relation to the filler/binder ratio, considering the filler as part of the binding agent rather than as an extension of the aggregate grading;

2. Effect of changes in binder viscosity, in order that the contribution of binder properties might be assessed. This could usefully be extended to include various types of binder, especially Trinidad Lake asphalt, high viscosity tars, bitumens and combinations thereof.

It is considered that the essential requirement at present is an assessment of the implications of the Marshall test, especially in whether high stability, or low flow, or a ratio of these parameters is the applicable criterion for predicting the suitability of a material for the construction of non-deforming layers under load. The Wheel-tracking test provides some indication of the probable behaviour in service and joint work on the Marshall test and the wheel-tracking test is recommended.

Work on relating the efflux rate of a graded sand to its behaviour as a constituent material of a sand asphalt has been confined to one particular aggregate grading. An extension of the work using aggregate of various gradings may lead to an expression
relating both the particle interaction and the fineness of the aggregate grading to the behaviour of sand asphalts. This aspect may warrant further investigation.

The higher Marshall stability value resulting from marked particle interaction presents the possibility of difficulty in field compaction, a laboratory workability test is therefore required in order that this may be assessed quantitatively. It is suggested that the relative density and the corresponding stability on a range of levels of compaction might present a possible approach to this problem. Preliminary work in developing this test might be performed with the use of oils with a viscosity at room temperature similar to that of the binder at compaction.

Conclusions drawn from the results of experimental work described in this thesis, with particular reference to the contribution of the constituent materials to the performance of the mixed material, are based on a limited number of specimens at specified conditions. Results from certain of the above recommendations would either validate the proposed theories, or provide a further insight into the physical structure and load bearing qualities of dense asphalt mixes.
Plate 7

THE WHEEL-TRACKING APPARATUS
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<th>Efflux times for 5 kg</th>
<th>Time for 100 kg</th>
<th>Efflux rate (cc/sec)</th>
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<tbody>
<tr>
<td>1-2</td>
<td>1.422</td>
<td>1.776</td>
<td>1.522</td>
</tr>
<tr>
<td>1.5</td>
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<td>1.776</td>
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<td>2</td>
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<td>1.776</td>
<td>1.522</td>
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<td>3</td>
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<tr>
<td>12</td>
<td>1.422</td>
<td>1.776</td>
<td>1.522</td>
</tr>
</tbody>
</table>

**NOTES.** To facilitate subsequent calculations, all values have been made to a greater accuracy than can normally be required.
### Table 2

**Efflux rates of sampled sized sands at 1/4" diameter orifice**

**a. Gr. Ives rounded flint gravel**

<table>
<thead>
<tr>
<th>Size (U.S. mesh no.)</th>
<th>S.G.</th>
<th>Weight (lb)</th>
<th>Efflux time (sec)</th>
<th>Time for 1 cm (sec)</th>
<th>Discharge rate (gpm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 - 14</td>
<td>2.69</td>
<td>11 kg</td>
<td>24.4</td>
<td>1.2</td>
<td>10</td>
</tr>
<tr>
<td>10 - 16</td>
<td>2.69</td>
<td>11 kg</td>
<td>31.0</td>
<td>2.9</td>
<td>7</td>
</tr>
<tr>
<td>16 - 25</td>
<td>2.69</td>
<td>11 kg</td>
<td>48.8</td>
<td>4.4</td>
<td>5</td>
</tr>
<tr>
<td>25 - 40</td>
<td>2.69</td>
<td>11 kg</td>
<td>54.9</td>
<td>7.8</td>
<td>3</td>
</tr>
<tr>
<td>40 - 65</td>
<td>2.69</td>
<td>11 kg</td>
<td>61.7</td>
<td>6.7</td>
<td>2</td>
</tr>
<tr>
<td>65 - 100</td>
<td>2.69</td>
<td>11 kg</td>
<td>72.1</td>
<td>8.1</td>
<td>1</td>
</tr>
<tr>
<td>6.45 kg 900 gms</td>
<td>11.5</td>
<td>5.7</td>
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</table>

**b. Thames Valley irregular flint gravel**

<table>
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<tr>
<th>Size (U.S. mesh no.)</th>
<th>S.G.</th>
<th>Weight (lb)</th>
<th>Efflux time (sec)</th>
<th>Time for 1 cm (sec)</th>
<th>Discharge rate (gpm/sec)</th>
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<tr>
<td>7 - 14</td>
<td>2.66</td>
<td>33 kg</td>
<td>3.5</td>
<td>4.1</td>
<td>32</td>
</tr>
<tr>
<td>10 - 16</td>
<td>2.75</td>
<td>33 kg</td>
<td>2.6</td>
<td>1.7</td>
<td>23</td>
</tr>
<tr>
<td>16 - 25</td>
<td>2.85</td>
<td>33 kg</td>
<td>3.5</td>
<td>4.1</td>
<td>22</td>
</tr>
<tr>
<td>25 - 40</td>
<td>2.95</td>
<td>33 kg</td>
<td>4.8</td>
<td>7.8</td>
<td>17</td>
</tr>
<tr>
<td>40 - 65</td>
<td>3.05</td>
<td>33 kg</td>
<td>6.0</td>
<td>9.0</td>
<td>12</td>
</tr>
<tr>
<td>65 - 100</td>
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<td>33 kg</td>
<td>7.2</td>
<td>11.2</td>
<td>9</td>
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<tr>
<td>6.45 kg 900 gms</td>
<td>11.5</td>
<td>5.7</td>
<td></td>
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# Afflux Rate of Single Sizes Range - Vibrating Vibrator

2a. Crushed Limestone, e.t. = 0.75

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<th>Weight</th>
<th>Afflux Time</th>
<th>Fraction</th>
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<td>642.0 gm</td>
<td>1.0</td>
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</tr>
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<td>10</td>
<td>462.5 gm</td>
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</tr>
<tr>
<td>14</td>
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<td>1.0</td>
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<td>18</td>
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<td>49.5</td>
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<td>25</td>
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<td>22.75 kg</td>
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<td>72</td>
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<td>100</td>
<td>28.25 kg</td>
<td>91.0</td>
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2b. Hardened Crushed Porphyr, e.t. = 0.29

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<td>454.0 gm</td>
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<tr>
<td>14</td>
<td>1085.0 gm</td>
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<td>1.0</td>
<td>1.0</td>
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<tr>
<td>52</td>
<td>3.5 kg</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>72</td>
<td>3.5 kg</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>100</td>
<td>3.5 kg</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
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TABLE 4

Effect of quantity of material in efflux test apparatus on rate of efflux. *D* = 1" diameter orifice.

4a. Helderini, Grade B. *D* = 2.97

<table>
<thead>
<tr>
<th>Weight (gm)</th>
<th>Method of filling</th>
<th>Time (sec)</th>
<th>Rate (cc/sec)</th>
</tr>
</thead>
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<td>1500</td>
<td>c</td>
<td>12.4</td>
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<td>2000</td>
<td>d</td>
<td>11.9</td>
<td>7.7</td>
</tr>
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<td>7.7</td>
</tr>
<tr>
<td>3000</td>
<td>a</td>
<td>11.9</td>
<td>7.7</td>
</tr>
</tbody>
</table>

Method of filling

a. Material poured from top of apparatus
b. Material placed by scoop from minimum height
c. Material poured as method (a) and cooled
d. Material poured as method (a) and apparatus inclined at 45°.

4b. Mountsorrel granite - Graded, S.G. = 2.64

<table>
<thead>
<tr>
<th>Weight (gm)</th>
<th>Time (sec)</th>
<th>Rate (cc/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>5.9</td>
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</tr>
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<td>17.9</td>
<td>1.1</td>
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<tr>
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<td>1.4</td>
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<tr>
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<td>1.4</td>
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</table>
Angle of repose of single stone materials

<table>
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<th>Angle of Repose</th>
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<tbody>
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<tr>
<td>18</td>
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<td>33</td>
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</table>

50. Eau Claire gravel

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<th>Angle of Repose</th>
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<td>10-12</td>
<td>40</td>
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<td>14-16</td>
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<td>39</td>
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<td>18-20</td>
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<td>35</td>
<td>33</td>
</tr>
<tr>
<td>25-30</td>
<td>32</td>
<td>31</td>
<td>29</td>
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</tbody>
</table>

54. Frontal drill crushed gravel

<table>
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<th>Right Hand Angle</th>
<th>Angle of Repose</th>
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</thead>
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<td>45</td>
<td>45</td>
</tr>
<tr>
<td>10-12</td>
<td>40</td>
<td>40</td>
<td>40</td>
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<tr>
<td>14-16</td>
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<tr>
<td>18-20</td>
<td>36</td>
<td>35</td>
<td>33</td>
</tr>
</tbody>
</table>
### Table 1: Test Results for Distilled Water

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Density (g/cm³)</th>
<th>Volume (cm³)</th>
<th>Mass (g)</th>
<th>Volume % Difference</th>
<th>Mass % Difference</th>
<th>Clarity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.234</td>
<td>12.3</td>
<td>15.1</td>
<td>0.134</td>
<td>0.084</td>
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<tr>
<td>2</td>
<td>1.245</td>
<td>12.4</td>
<td>15.2</td>
<td>0.145</td>
<td>0.075</td>
<td>Yes</td>
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<tr>
<td>3</td>
<td>1.256</td>
<td>12.5</td>
<td>15.3</td>
<td>0.156</td>
<td>0.066</td>
<td>Yes</td>
</tr>
<tr>
<td>...</td>
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<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
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<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 2: Test Results for Distilled Water

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Height (mm)</th>
<th>Mass (g)</th>
<th>Volume (cm³)</th>
<th>Mass % Difference</th>
<th>Volume % Difference</th>
<th>Clarity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<tr>
<td>2</td>
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<td>12.4</td>
<td>0.075</td>
<td>0.145</td>
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<tr>
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<td>12.5</td>
<td>0.066</td>
<td>0.156</td>
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<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
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</tr>
<tr>
<td>Average</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
TABLE 7 (continued)

Typical Marshall test results for stone filled samples

7c. Gravel stone - Gravel sand.  Binder content = 5.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Height (in)</th>
<th>a.s.</th>
<th>Voids (%)</th>
<th>Voids filled (per cent)</th>
<th>D</th>
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<tbody>
<tr>
<td>211</td>
<td></td>
<td>2.419</td>
<td>6.616</td>
<td>74.77</td>
<td>14</td>
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<tr>
<td>212</td>
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<td>2.314</td>
<td>8.64</td>
<td>74.79</td>
<td>1450</td>
</tr>
<tr>
<td>237</td>
<td>2 7/16</td>
<td>2.316</td>
<td>7.76</td>
<td>75.98</td>
<td>1350</td>
</tr>
<tr>
<td>238</td>
<td></td>
<td>2.224</td>
<td>4.96</td>
<td>74.59</td>
<td>1340</td>
</tr>
<tr>
<td>239</td>
<td>2 7/16</td>
<td>2.302</td>
<td>4.32</td>
<td>74.70</td>
<td>1370</td>
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<td>4.607</td>
<td>77.87</td>
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<td>4.372</td>
<td>77.77</td>
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<tr>
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<td>4.576</td>
<td>77.47</td>
<td>1410</td>
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<tr>
<td>266</td>
<td></td>
<td>2.300</td>
<td>4.956</td>
<td>74.19</td>
<td>1410</td>
</tr>
<tr>
<td>Average</td>
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<td>2.305</td>
<td>4.20</td>
<td>75.47</td>
<td>1410</td>
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### Table 5
Summary of Marshall test results for stone fillers results

<table>
<thead>
<tr>
<th>Binder</th>
<th>Number of Stability Flows (bk)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
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<tr>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Binder</th>
<th>Number of Stability Flows (bk)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
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<tr>
<td>3</td>
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<tr>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
</tr>
</tbody>
</table>

### Table 6b
Granite stone - gravel sand

<table>
<thead>
<tr>
<th>Binder</th>
<th>Number of Stability Flows (bk)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>4</td>
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<tr>
<td>2</td>
<td>2</td>
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<tr>
<td>3</td>
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<tr>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Binder</th>
<th>Number of Stability Flows (bk)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
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<tr>
<td>3</td>
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<tr>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
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</tbody>
</table>
### Gravel stone - Crushed gravel

<table>
<thead>
<tr>
<th>Binder Content (%)</th>
<th>Number of Specimens</th>
<th>Stability</th>
<th>Flow (x0.1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>6</td>
<td>1400</td>
<td>1.3</td>
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<td>4</td>
<td>6</td>
<td>1350</td>
<td>1.2</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>1300</td>
<td>1.1</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>1250</td>
<td>1.0</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>1200</td>
<td>0.9</td>
</tr>
<tr>
<td>8</td>
<td>6</td>
<td>1150</td>
<td>0.8</td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>1100</td>
<td>0.7</td>
</tr>
</tbody>
</table>

### Gravel stone - Granite sand

<table>
<thead>
<tr>
<th>Binder Content (%)</th>
<th>Number of Specimens</th>
<th>Stability</th>
<th>Flow (x0.1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>6</td>
<td>1400</td>
<td>1.3</td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>1350</td>
<td>1.2</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>1300</td>
<td>1.1</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>1250</td>
<td>1.0</td>
</tr>
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<td>7</td>
<td>6</td>
<td>1200</td>
<td>0.9</td>
</tr>
<tr>
<td>8</td>
<td>6</td>
<td>1150</td>
<td>0.8</td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>1100</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Flow equals dry.
### Table 9

Summary of Marshall test results for stone filled asphaltic concrete aggregate, varying stone contents.

Granite aggregate throughout, 3% filler.

#### 3a. 30% stone

<table>
<thead>
<tr>
<th>Binder Number (%)</th>
<th>Number of specimens</th>
<th>Stability</th>
<th>Flow (lb)</th>
<th>M. V. (lb/ft³)</th>
<th>W. V. (lb/ft³)</th>
<th>A. V. (lb/ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2</td>
<td>1715</td>
<td>11</td>
<td>4.148</td>
<td>14.72</td>
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</tr>
<tr>
<td>4a</td>
<td>4</td>
<td>1620</td>
<td>11</td>
<td>4.171</td>
<td>14.7</td>
<td>48.82</td>
</tr>
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<td>11</td>
<td>4.176</td>
<td>14.77</td>
<td>48.86</td>
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<td>4</td>
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<td>11</td>
<td>4.159</td>
<td>14.92</td>
<td>48.91</td>
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<td>4</td>
<td>1750</td>
<td>11</td>
<td>4.195</td>
<td>14.87</td>
<td>48.79</td>
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<td>12</td>
<td>4.199</td>
<td>14.83</td>
<td>48.77</td>
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<td>8</td>
<td>2005</td>
<td>12</td>
<td>4.212</td>
<td>16.09</td>
<td>48.61</td>
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<tr>
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<td>2005</td>
<td>13</td>
<td>4.217</td>
<td>16.06</td>
<td>48.63</td>
</tr>
<tr>
<td>8</td>
<td>9</td>
<td>1950</td>
<td>16</td>
<td>4.274</td>
<td>16.28</td>
<td>49.56</td>
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<td>9</td>
<td>2</td>
<td>1550</td>
<td>21</td>
<td>4.394</td>
<td>17.96</td>
<td>49.73</td>
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</table>

#### 3b. 40% stone

<table>
<thead>
<tr>
<th>Binder Number (%)</th>
<th>Number of specimens</th>
<th>Stability</th>
<th>Flow (lb)</th>
<th>M. V. (lb/ft³)</th>
<th>W. V. (lb/ft³)</th>
<th>A. V. (lb/ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>2</td>
<td>1640</td>
<td>11</td>
<td>4.160</td>
<td>14.79</td>
<td>48.97</td>
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<td>3a</td>
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<td>1600</td>
<td>11</td>
<td>4.182</td>
<td>14.76</td>
<td>48.94</td>
</tr>
<tr>
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<td>3</td>
<td>1790</td>
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<td>4.160</td>
<td>14.72</td>
<td>48.91</td>
</tr>
<tr>
<td>4a</td>
<td>3</td>
<td>1790</td>
<td>11</td>
<td>4.163</td>
<td>14.73</td>
<td>48.91</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>1790</td>
<td>11</td>
<td>4.164</td>
<td>14.73</td>
<td>48.92</td>
</tr>
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<td>1790</td>
<td>11</td>
<td>4.166</td>
<td>14.74</td>
<td>48.93</td>
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<td>3</td>
<td>1800</td>
<td>11</td>
<td>4.178</td>
<td>14.79</td>
<td>48.98</td>
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<td>4.179</td>
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<td>11</td>
<td>4.222</td>
<td>16.18</td>
<td>49.77</td>
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<td>1950</td>
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<td>4.224</td>
<td>16.19</td>
<td>49.78</td>
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<td>1900</td>
<td>11</td>
<td>4.246</td>
<td>16.25</td>
<td>49.83</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>1550</td>
<td>21</td>
<td>4.394</td>
<td>17.96</td>
<td>49.73</td>
</tr>
</tbody>
</table>

**Note:** For results of 50% stone content, refer to table 3b.
Table 4 (continued)

Summary of small test results for stone fill specimen: Continue aggregate grading, vary the stone content. Gravel aggregate throughout. Fuller.

3c. CO stone

<table>
<thead>
<tr>
<th>Binder</th>
<th>Number of content (l)</th>
<th>specimens</th>
<th>(lb)</th>
<th>(in. &quot; )</th>
</tr>
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<tr>
<td>4</td>
<td>4</td>
<td>1540</td>
<td>12</td>
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<tr>
<td>4.5</td>
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<td>-</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>1560</td>
<td>12</td>
<td>-</td>
</tr>
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<td>6</td>
<td>1570</td>
<td>12</td>
<td>-</td>
</tr>
<tr>
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<td>-</td>
</tr>
<tr>
<td>7.5</td>
<td>2</td>
<td>1950</td>
<td>12</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: Heights of specimens of CO stone content (in. " ) varied from 12 at 4 lb to 12 at 7.5 lb.
TABLE 10

Summary of Marshall test results for stone filled asphalt

I.a. gravelly aggregates

10a. Continuously graded granite stone (60%)
Continuously graded gravel sand with 7-15 fraction omitted.
7% filler

<table>
<thead>
<tr>
<th>Binder</th>
<th>Number of Specimens</th>
<th>Stability (lb)</th>
<th>Flow (cm)</th>
<th>Voids (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2</td>
<td>1375</td>
<td>10</td>
<td>15</td>
</tr>
<tr>
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10b. Single sizes granite stone (50%)
"asphalt" sand
7% filler

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<th>Number of Specimens</th>
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216.
### TABLE 11

Summary of Lab results for granite: Mound Sorrel crushed granite.

**Continuous aggregate grading: 49.84% filler.**

#### 11a. Mound Sorrell crushed granite

<table>
<thead>
<tr>
<th>Binder Content (%)</th>
<th>Number of Specimens</th>
<th>Stability (lb)</th>
<th>Flow (x0.01&quot;)</th>
<th>Voids (%)</th>
<th>Voids Filled (%)</th>
<th>Aggregate (%)</th>
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<tbody>
<tr>
<td>4</td>
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#### 11b. Thames Valley irregular flint gravel

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<th>Stability (lb)</th>
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<th>Voids (%)</th>
<th>Voids Filled (%)</th>
<th>Aggregate (%)</th>
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"*"
Table 11 (continued)

Summary of Marshall test results for sand asphalt

11a. Croby granite

<table>
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<tr>
<th>Binder content (%) specimens</th>
<th>Number of specimens</th>
<th>Stability (lb)</th>
<th>Flow (x0.01&quot;)</th>
<th>S.B. Voids (%)</th>
<th>Voids Air Voids ratio in mix</th>
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<tbody>
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11b. Crushed limestone

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<th>Stability (lb)</th>
<th>Flow (x0.01&quot;)</th>
<th>S.B. Voids (%)</th>
<th>Voids Air Voids ratio in mix</th>
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11c. St. Ives rounded flint gravel

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<th>Voids Air Voids ratio in mix</th>
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<td>Flow (x100)</td>
<td>Value</td>
<td>Weight (%)</td>
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**Table 11 (continued)**

**Summary of Stability Test Results for Various Binder Contents**

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<th>Number of Specimens</th>
<th>Stability (lb)</th>
<th>Flow (x100)</th>
<th>Value</th>
<th>Weight (%)</th>
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<td>10</td>
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<td></td>
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<td>9</td>
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<td>17</td>
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**Table 11a**

**Summary of Stability Test Results for Various Binder Contents**

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**Table 11b**

**Summary of Stability Test Results for Various Binder Contents**

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**Table 11c**

**Summary of Stability Test Results for Various Binder Contents**

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<th>Flow (x100)</th>
<th>Value</th>
<th>Weight (%)</th>
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11b. Leighton Hazard round: No. 14
APPENDIX 7. Computed results in unloading of discharge of granular mixture.
### Table 1

**Critic: Sieve and Ballotini sizes**

1. **Grades** - measured with internal calipers and micrometer, average of 10 readings per sieve.

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<th>Nominal diameter</th>
<th>Measured diameter</th>
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2. **Ballotini sizes** - measured with micrometer, average of 10 readings on each of its three planes.

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<th>Ballotini size</th>
<th>Measured (mm)</th>
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</tr>
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<td>0.6 mm</td>
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<td>0.5 mm</td>
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<tr>
<td>11</td>
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<td>0.2 mm</td>
</tr>
<tr>
<td>16</td>
<td>0.1 mm</td>
</tr>
</tbody>
</table>

3. **J.S. Sieve sizes** - taken as the real and rounded sieve sizes of charter, as per table.

<table>
<thead>
<tr>
<th>Sieve size range (mm)</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
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<tr>
<td>6</td>
<td>20</td>
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<td>8</td>
<td>14</td>
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<td>10</td>
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<td>12</td>
<td>7</td>
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<td>14</td>
<td>9</td>
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<tr>
<td>16</td>
<td>6</td>
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<tr>
<td>18</td>
<td>4</td>
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<td>20</td>
<td>3</td>
</tr>
<tr>
<td>22</td>
<td>2</td>
</tr>
<tr>
<td>24</td>
<td>1</td>
</tr>
</tbody>
</table>
APPENDIX

Efflux rate of steel ball from the column mixture

a) Material - 3/4" steel ball broken; no inconsistency in size or weight; 0.015" micrometer measurement.

b) Size - measured diameter = 0.75".

c) Specific gravity = measured value to be 7.86.

d) Efflux time = for 100%, efflux time = 2.5 sec.

efflux rate = 4.11 cc/sec.

This efflux rate is based on research which shows that 4.11 cc/sec. time for the efflux of a known weight of steel ball to be 0.175" diameter. Therefore, we can assume the efflux rate will be in the range of 0.175"-0.250" diameter rate.
\[ \Pi_1 \quad \Phi(\rho \omega, \cdot) \quad \Theta = \frac{L}{D} \quad \Omega \]

\[ \Pi_2 \quad \Phi(\rho \omega, \cdot) \quad \Theta = \frac{L}{D} \quad \Omega \]

\[ \Pi_3 \quad \Phi(\rho \omega, \cdot) \quad \Theta = \frac{L}{D} \quad \Omega \]

\[ \Pi_4 \quad \Phi(\rho \omega, \cdot) \quad \Theta = \frac{L}{D} \quad \Omega \]
\[ f(x, z) \] and \[ g(z) \]

\[ \frac{M}{p^{2} \rho E} = R \left[ (\sigma), (\frac{\sigma}{E}), (\frac{\sigma}{E}), (\frac{\sigma}{E}) \right] \]

**Elimination of extraneous variables**

**H** - It has been shown in Table 5 that the bulk modulus is independent of the kind of material in \[ A \] or \[ \Delta \].

**\( \Delta \)** - Moore and Trowbridge have shown that the \[ \Delta \] discharge is independent of the kind of material in \[ A \]. In this case, \[ \Delta \] = 0.07 for the Jorge test.

**C** - Shear box and vane tests provided evidence that the finest size fractions of all samples, starting sizes and in the 15 (silt).

**\( \Theta \)** - The theoretical maximum shear constant is in a single.

**Coefficient of friction**

The angle of friction \( \phi \) is the angle at which a material has been found to fail in shear in a single test. For the Jorge test:

\[ \theta \phi \]

For particles of equal size, a factor of porosity \( \nu \) and shape \( \sigma \), the resistance of a wall will increase as the particle...
component particles

If the number of particles is

\[ \bar{N} = (n_1, x_1, \ldots, x_n) \]

then

\[ \bar{N} = (n_1, x_1, \ldots, x_n) \]

or

\[ \bar{N} = (n_1, x_1, \ldots, x_n) \]

hence, if \( \bar{x} \) is the number of particles of size \( x \), then

\[ \bar{N} = (n_1, x_1, \ldots, x_n) \]

or

\[ \bar{N} = (n_1, x_1, \ldots, x_n) \]

now \( \bar{x} \) is the proportion of particles, expressed as a fraction of the total number of particles.

\[ \bar{N} = (n_1, x_1, \ldots, x_n) \]

or

\[ \bar{N} = (n_1, x_1, \ldots, x_n) \]

or

\[ \bar{N} = (n_1, x_1, \ldots, x_n) \]

where

- \( P = \text{proportion of number of particles} \)
- \( \Pi \) denotes product
- \( \sum \) denotes summation
- \( a \) = weight of each particle
- \( W \) = weight of total number of particles (constant)
- \( a_i \) = weight of each particle size \( i \)

then

\[ \log \bar{N} = \frac{1}{n} \sum_{i=1}^{n} a_i \log a_i \]

Ref. a = I, II, ..., n. F. = Particle size measurement, interpretation, and application.
Verification of method of determining equivalent particle size using the efflux test apparatus.

\[ \log F = \sum_{i=1}^{n} w_i \log N_i \]

\[ \frac{w}{v} \]

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assuming that time A (sec/100cc) = \frac{gm}{2.449 \times 2.785} \times \text{time} + \text{rate} \\

then \quad \frac{2}{\text{rate A}} = \frac{1}{\text{rate B}} + \frac{1}{\text{rate C}} \\

<table>
<thead>
<tr>
<th>size range</th>
<th>Efflux rates (cc/sec)</th>
<th>Theoretical N/S</th>
<th>L.B. efflux rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 - 18</td>
<td>17.0</td>
<td>32.3</td>
<td>32.3</td>
</tr>
<tr>
<td>18 - 25</td>
<td>19.1</td>
<td>34.7</td>
<td>34.7</td>
</tr>
<tr>
<td>25 - 36</td>
<td>20.7</td>
<td>36.4</td>
<td>36.4</td>
</tr>
<tr>
<td>36 - 52</td>
<td>21.7</td>
<td>37.2</td>
<td>37.2</td>
</tr>
<tr>
<td>52 - 72</td>
<td>22.2</td>
<td>37.6</td>
<td>37.6</td>
</tr>
<tr>
<td>72 - 100</td>
<td>19.9</td>
<td>36.3</td>
<td>36.3</td>
</tr>
</tbody>
</table>

The efflux rate of an aggregate combination can therefore be determined from the proportion and efflux rates of the constituent materials.
The specific gravity of voids in mix

\[ G_v = \frac{G_m - G_t}{G_t} \times 100 \]

de where \( G_m \) = specific gravity of mix, \( G_t \) = theoretical specific gravity of voidless material, and \( G_v \) = specific gravity of voids.

then, volume of particle/100 gm of mix = \( \frac{1}{G_m} \)

\[ \therefore \text{ total volume of 100 gm of mix = } \frac{1}{G_m} \]

hence the specific gravity of the voids.

The specific gravity of the voidless mix can be obtained in a similar manner by computing the volume of material per 100 gm of mix.

b) Voids in mix

if \( G_A \) = specific gravity of mix
\( G_n \) = theoretical specific gravity of voidless material
\( V \) = volume of 100 gm of mix
\( V_s \) = volume of 100 gm of voidless material
\( V_v \) = volume of voids / 100 gm of mix

then, voids in mix = \( \frac{V_v}{V} \times 100 = V_s - V \times 100 \)

now \( V = \frac{100}{G_A} \) and \( V_s = \frac{100}{G_n} \)

\[ \therefore \text{ voids in mix (\( V_v \)) = } \frac{G_n - G_A}{G_n} \times 100 \]
c) **Voids in mixed aggregate**

If

- \( V_B \) = specific gravity of binder
- \( G_A \) = specific gravity of mix
- \( V_B \) = volume of binder / 100 gm of mix
- \( V_A \) = volume of aggregate / 100 gm of mix
- \( V_V \) = volume of voids / 100 gm of mix
- \( V \) = volume of 100 gm of mix
- \( W_B \) = weight of binder / 100 gm of mix

then % voids in mixed aggregate (V.M.A.) = \[ \frac{V_B + V_V}{V} \]

\[ = \frac{V_B}{V} + \frac{V_V}{V} \]

hence \( V.M.A. = \frac{V_B}{G_A} \cdot \frac{G_A + V}{G_B} \times 100 \)

where \( V_m \) = voids in mix (see previous section).

d) **Aggregate voids filled with binder**

% aggregate voids filled with binder = \[ \frac{V_B}{V} \times \frac{100}{V.M.A.} \]

\[ = \frac{V_B}{V.T.A. \times V} \times 100 \]

\[ = \frac{V_B}{V.T.A.} \cdot \frac{G_A}{G_B} \times 100 \]

\[ = \frac{V.B}{V.T.A.} \]
**APPENDIX 7**

Computed results in analysis of rate of discharge of granular materials from an orifice.

a. Ballotini: 3", 4" and 2" diameter orifices.

evaluation of functions \( \frac{V}{D^{2.32}g^2} = \left( \frac{D}{d} - 3 \right) \) (see Figure 4)

**i) 3" diameter orifice** \( D = 9.245 \text{ mm} \)

<table>
<thead>
<tr>
<th>Size (mm)</th>
<th>( d ) (mm)</th>
<th>( V ) (mm(^3)/sec)</th>
<th>( \log \frac{V}{D^{2.32}g^2} )</th>
<th>( \log \left( \frac{D}{d} - 3 \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1(\frac{1}{2})-2mm.</td>
<td>2.094</td>
<td>4.174 (\times) 10(3)</td>
<td>T. 2093</td>
<td>0.7088</td>
</tr>
<tr>
<td>Grade 4</td>
<td>1.105</td>
<td>6.263 &quot;</td>
<td>T. 3561</td>
<td>0.1095</td>
</tr>
<tr>
<td>5</td>
<td>0.807</td>
<td>6.935 &quot;</td>
<td>T. 4503</td>
<td>0.9397</td>
</tr>
<tr>
<td>8</td>
<td>0.472</td>
<td>8.594 &quot;</td>
<td>T. 5232</td>
<td>1.1085</td>
</tr>
<tr>
<td>10</td>
<td>0.297</td>
<td>9.791 &quot;</td>
<td>T. 5957</td>
<td>1.4053</td>
</tr>
<tr>
<td>11</td>
<td>0.232</td>
<td>10.048 &quot;</td>
<td>T. 5913</td>
<td>1.5677</td>
</tr>
<tr>
<td>14</td>
<td>0.127</td>
<td>9.654 &quot;</td>
<td>T. 5746</td>
<td>1.2456</td>
</tr>
<tr>
<td>18</td>
<td>0.072</td>
<td>8.954 &quot;</td>
<td>T. 5235</td>
<td>2.1653</td>
</tr>
</tbody>
</table>

**ii) 4" diameter orifice** \( D = 15.875 \text{ mm} \)

<table>
<thead>
<tr>
<th>Size (mm)</th>
<th>( d ) (mm)</th>
<th>( V ) (mm(^3)/sec)</th>
<th>( \log \frac{V}{D^{2.32}g^2} )</th>
<th>( \log \left( \frac{D}{d} - 3 \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3mm</td>
<td>3.098</td>
<td>20.33 (\times) 10(3)</td>
<td>T. 3105</td>
<td>0.772</td>
</tr>
<tr>
<td>1(\frac{1}{2})-2mm.</td>
<td>2.094</td>
<td>23.96 &quot;</td>
<td>T. 3818</td>
<td>0.6110</td>
</tr>
<tr>
<td>Grade 4</td>
<td>1.105</td>
<td>30.40 &quot;</td>
<td>T. 4552</td>
<td>1.0088</td>
</tr>
<tr>
<td>5</td>
<td>0.807</td>
<td>32.79 &quot;</td>
<td>T. 5617</td>
<td>1.2220</td>
</tr>
<tr>
<td>8</td>
<td>0.472</td>
<td>37.96 &quot;</td>
<td>T. 5817</td>
<td>1.4958</td>
</tr>
<tr>
<td>10</td>
<td>0.297</td>
<td>41.65 &quot;</td>
<td>T. 6156</td>
<td>1.7029</td>
</tr>
<tr>
<td>11</td>
<td>0.232</td>
<td>41.22 &quot;</td>
<td>T. 6164</td>
<td>1.8158</td>
</tr>
<tr>
<td>14</td>
<td>0.127</td>
<td>38.84 &quot;</td>
<td>T. 5916</td>
<td>2.0664</td>
</tr>
<tr>
<td>18</td>
<td>0.072</td>
<td>34.94 &quot;</td>
<td>T. 5457</td>
<td>2.3775</td>
</tr>
</tbody>
</table>

**iii) 2" diameter orifice** \( D = 23.29 \text{ mm} \)

<table>
<thead>
<tr>
<th>Size (mm)</th>
<th>( d ) (mm)</th>
<th>( V ) (mm(^3)/sec)</th>
<th>( \log \frac{V}{D^{2.32}g^2} )</th>
<th>( \log \left( \frac{D}{d} - 3 \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3mm</td>
<td>3.098</td>
<td>64.78 (\times) 10(3)</td>
<td>T. 3976</td>
<td>0.6549</td>
</tr>
<tr>
<td>1(\frac{1}{2})-2mm.</td>
<td>2.094</td>
<td>74.20 &quot;</td>
<td>T. 4566</td>
<td>0.9097</td>
</tr>
<tr>
<td>Grade 4</td>
<td>1.105</td>
<td>83.48 &quot;</td>
<td>T. 5420</td>
<td>1.2572</td>
</tr>
<tr>
<td>5</td>
<td>0.807</td>
<td>94.55 &quot;</td>
<td>T. 5618</td>
<td>1.4127</td>
</tr>
<tr>
<td>8</td>
<td>0.472</td>
<td>102.6 &quot;</td>
<td>T. 6115</td>
<td>1.6660</td>
</tr>
<tr>
<td>10</td>
<td>0.297</td>
<td>112.6 &quot;</td>
<td>T. 6376</td>
<td>1.8775</td>
</tr>
<tr>
<td>11</td>
<td>0.232</td>
<td>117.7 &quot;</td>
<td>T. 6420</td>
<td>1.9886</td>
</tr>
<tr>
<td>14</td>
<td>0.127</td>
<td>103.8 &quot;</td>
<td>T. 6107</td>
<td>2.2568</td>
</tr>
<tr>
<td>18</td>
<td>0.072</td>
<td>95.24 &quot;</td>
<td>T. 5650</td>
<td>2.5058</td>
</tr>
</tbody>
</table>
b. Ballotini : *\n*, *\n* and *\n* diameter orifices

**evaluation of function of friction (see figure 8)**

\[
\log \frac{V}{D^2 g^2} = \log k \left( \frac{D}{d} - 3 \right) - \log X
\]

i) *\n* diameter orifice \( \left( \frac{V}{D^2 g^2} \right) = 0.145 \left( \frac{D}{d} - 3 \right) ^{0.3} \)

<table>
<thead>
<tr>
<th>Size</th>
<th>d (mm)</th>
<th>V (mm³/sec)</th>
<th>log 0.145(D/d)</th>
<th>log V/D²g²</th>
<th>log X</th>
<th>log 1/X</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>0.072</td>
<td>8.954.10²</td>
<td>T. 7925</td>
<td>T. 5835</td>
<td>T. 10</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>0.127</td>
<td>9.654 &quot;</td>
<td>T. 7162</td>
<td>T. 5740</td>
<td>0.146</td>
<td>T. 94</td>
</tr>
<tr>
<td>11</td>
<td>0.232</td>
<td>10.048 &quot;</td>
<td>T. 6329</td>
<td>T. 5913</td>
<td>0.1416</td>
<td>T. 84</td>
</tr>
<tr>
<td>10</td>
<td>0.297</td>
<td>9.691 &quot;</td>
<td>T. 5977</td>
<td>T. 5757</td>
<td>0.1420</td>
<td>T. 84</td>
</tr>
<tr>
<td>8</td>
<td>0.472</td>
<td>8.594 &quot;</td>
<td>T. 5299</td>
<td>T. 5235</td>
<td>0.0634</td>
<td>T. 74</td>
</tr>
</tbody>
</table>

ii) *\n* diameter orifice \( \left( \frac{V}{D^2 g^2} \right) = 0.165 \left( \frac{D}{d} - 3 \right) ^{0.5} \)

<table>
<thead>
<tr>
<th>Size</th>
<th>d (mm)</th>
<th>V (mm³/sec)</th>
<th>log 0.165(D/d)</th>
<th>log V/D²g²</th>
<th>log X</th>
<th>log 1/X</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>0.072</td>
<td>34.94.10²</td>
<td>T. 8014</td>
<td>T. 5457</td>
<td>T. 10</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>0.127</td>
<td>38.84 &quot;</td>
<td>T. 7566</td>
<td>T. 5916</td>
<td>0.1470</td>
<td>T. 10</td>
</tr>
<tr>
<td>11</td>
<td>0.232</td>
<td>41.22 &quot;</td>
<td>T. 6709</td>
<td>T. 6154</td>
<td>0.0645</td>
<td>T. 10</td>
</tr>
<tr>
<td>10</td>
<td>0.297</td>
<td>41.05 &quot;</td>
<td>T. 6427</td>
<td>T. 6154</td>
<td>0.0272</td>
<td>T. 10</td>
</tr>
<tr>
<td>8</td>
<td>0.472</td>
<td>37.96 &quot;</td>
<td>T. 5885</td>
<td>T. 5817</td>
<td>0.0648</td>
<td>T. 10</td>
</tr>
</tbody>
</table>

iii) *\n* diameter orifice \( \left( \frac{V}{D^2 g^2} \right) = 0.180 \left( \frac{D}{d} - 3 \right) ^{0.25} \)

<table>
<thead>
<tr>
<th>Size</th>
<th>d (mm)</th>
<th>V (mm³/sec)</th>
<th>log 0.180(D/d)</th>
<th>log V/D²g²</th>
<th>log X</th>
<th>log 1/X</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>0.072</td>
<td>95.24.10²</td>
<td>T. 8138</td>
<td>T. 5650</td>
<td>T. 10</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>0.127</td>
<td>105.8 &quot;</td>
<td>T. 7596</td>
<td>T. 6107</td>
<td>0.1489</td>
<td>T. 10</td>
</tr>
<tr>
<td>11</td>
<td>0.232</td>
<td>113.7 &quot;</td>
<td>T. 6994</td>
<td>T. 6420</td>
<td>0.0574</td>
<td>T. 10</td>
</tr>
<tr>
<td>10</td>
<td>0.297</td>
<td>112.6 &quot;</td>
<td>T. 6742</td>
<td>T. 6376</td>
<td>0.0366</td>
<td>T. 10</td>
</tr>
<tr>
<td>8</td>
<td>0.472</td>
<td>106.0 &quot;</td>
<td>T. 6268</td>
<td>T. 6115</td>
<td>0.0153</td>
<td>T. 10</td>
</tr>
</tbody>
</table>

iv) Angles of repose of Ballotini taken as a measure of the coefficient of friction (\( \mu \))

<table>
<thead>
<tr>
<th>Size</th>
<th>( \mu (°) )</th>
<th>log</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade 18</td>
<td>27.5</td>
<td>T. 7165</td>
</tr>
<tr>
<td>14</td>
<td>25</td>
<td>0.466 T. 6687</td>
</tr>
<tr>
<td>11</td>
<td>23</td>
<td>0.424 T. 6279</td>
</tr>
<tr>
<td>10</td>
<td>22</td>
<td>0.414 T. 6172</td>
</tr>
<tr>
<td>8</td>
<td>22</td>
<td>0.404 T. 6064</td>
</tr>
</tbody>
</table>

235.
c. single sized sands; ½" diameter orifice

evaluation of function of shape (see figure 10)

\[ V = 2.02 \left( \frac{D}{d} - 3 \right) \cdot \frac{f(z)}{Z^{2.3}} \]

particle size \( d = 0.724 \) \( \Rightarrow \left( \frac{D}{d} - 3 \right) = 18.91 \)

\[ \Rightarrow \frac{V \cdot Z^{2.3}}{4.213} = f(z) = f(\text{Voids} -33) \]

<table>
<thead>
<tr>
<th>Sand</th>
<th>( V ) (cc/sec)</th>
<th>Voids</th>
<th>( Z )</th>
<th>( \log \frac{V \cdot Z^{2.3}}{4.213} )</th>
<th>( \log Z )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ballotini</td>
<td>34.2</td>
<td>0.404</td>
<td>37.2</td>
<td>4.2</td>
<td>0.0029</td>
</tr>
<tr>
<td>St. Ives</td>
<td>26.7</td>
<td>0.554</td>
<td>42.5</td>
<td>9.5</td>
<td>0.2126</td>
</tr>
<tr>
<td>Thames Valley</td>
<td>25.0</td>
<td>0.589</td>
<td>42.6</td>
<td>9.6</td>
<td>0.2348</td>
</tr>
<tr>
<td>Limestone</td>
<td>21.0</td>
<td>0.700</td>
<td>47.1</td>
<td>14.1</td>
<td>0.3416</td>
</tr>
<tr>
<td>Harden</td>
<td>19.8</td>
<td>0.755</td>
<td>48.4</td>
<td>15.4</td>
<td>0.3594</td>
</tr>
<tr>
<td>Mountsorrel</td>
<td>19.1</td>
<td>0.810</td>
<td>49.9</td>
<td>16.9</td>
<td>0.4457</td>
</tr>
</tbody>
</table>

\[ \text{d. graded sands; ½" diameter orifice} \]

\[ \text{evaluation of function of shape (see figure 12)} \]

\[ \text{equivalent particle size} \quad d = 0.576 \] \( \Rightarrow \left( \frac{D}{d} - 3 \right) = 24.55 \]

\[ \Rightarrow \frac{V \cdot Z^{2.3}}{4.486} = f(z) = f(\text{Voids} -24) \]

<table>
<thead>
<tr>
<th>Sand</th>
<th>( V ) (cc/sec)</th>
<th>Voids</th>
<th>( Z )</th>
<th>( \log \frac{V \cdot Z^{2.3}}{4.486} )</th>
<th>( \log Z )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ballotini</td>
<td>44.0</td>
<td>0.466</td>
<td>26.5</td>
<td>2.5</td>
<td>0.2296</td>
</tr>
<tr>
<td>St. Ives</td>
<td>29.8</td>
<td>0.754</td>
<td>31.6</td>
<td>7.6</td>
<td>0.5396</td>
</tr>
<tr>
<td>Leighton Buzzard</td>
<td>29.1</td>
<td>0.754</td>
<td>33.3</td>
<td>9.3</td>
<td>0.5293</td>
</tr>
<tr>
<td>Thames Valley</td>
<td>27.4</td>
<td>0.810</td>
<td>33.3</td>
<td>9.3</td>
<td>0.5752</td>
</tr>
<tr>
<td>T.V.</td>
<td>25.7</td>
<td>0.885</td>
<td>34.7</td>
<td>10.7</td>
<td>0.6356</td>
</tr>
<tr>
<td>2T.V.</td>
<td>23.9</td>
<td>0.983</td>
<td>35.4</td>
<td>11.4</td>
<td>0.7090</td>
</tr>
<tr>
<td>Limestone</td>
<td>23.6</td>
<td>1.018</td>
<td>36.5</td>
<td>12.5</td>
<td>0.7385</td>
</tr>
<tr>
<td>1/2 T.V. - 1/2 M/S</td>
<td>22.9</td>
<td>1.000</td>
<td>36.9</td>
<td>12.9</td>
<td>0.7079</td>
</tr>
<tr>
<td>Harden</td>
<td>22.8</td>
<td>1.036</td>
<td>38.2</td>
<td>14.2</td>
<td>0.7410</td>
</tr>
<tr>
<td>Mountsorrel</td>
<td>21.3</td>
<td>1.036</td>
<td>39.6</td>
<td>15.6</td>
<td>0.7115</td>
</tr>
<tr>
<td>Basalt</td>
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<td>1.036</td>
<td>39.6</td>
<td>15.6</td>
<td>0.7094</td>
</tr>
<tr>
<td>Londerby</td>
<td>20.9</td>
<td>1.072</td>
<td>36.2</td>
<td>14.2</td>
<td>0.7380</td>
</tr>
<tr>
<td>Groby</td>
<td>20.0</td>
<td>1.072</td>
<td>40.3</td>
<td>16.3</td>
<td>0.7188</td>
</tr>
</tbody>
</table>

236.