USE OF NON-STANDARD ROAD AGGREGATES FROM WANGANUI AND TARANAKI REGIONS

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ABSTRACT

Road aggregates in the Wanganui and Taranaki regions are generally softer than those used elsewhere in New Zealand. However the costs of importing premium quality aggregate (e.g. that which complies with TNZ M/4 specification) is prohibitive. Instead, local practitioners have utilised local materials with success.

The report identifies the engineering properties of three principal rock types used in these regions and describes how aggregate production can be controlled, particularly by the use of the particle size distribution graph. It also provides methods for the control of pavement construction and describes some special techniques required to ensure long-term stability of the pavement.

1. INTRODUCTION

This report contains information to assist in evaluating the quality of rock deposits and the production and placement methods for non-standard aggregates available for road pavements in the Wanganui and Taranaki regions. These aggregates are generally softer than those normally accepted for road construction and many may not meet the Sand Equivalent requirements contained in TNZ[1] M/4 specification[2]. It has been traditional in roading construction over the last 35 years to specify a premium grade material (for example TNZ M/4) but dwindling resources of such materials, environmental restraints and high cartage costs encourage the investigation of rock types which produce marginal or non-standard roading aggregates.

As each rock type produces an aggregate having specific engineering characteristics, experimental work is necessary before a definite end product specification can be formulated. This report will help formulate a method for specification. A higher degree of risk of pavement failure must be accepted when non-standard aggregate is used in preference to premium quality material. Non-standard aggregates are regarded as inferior to premium quality basecourse and are not able to replace it where a pavement is heavily loaded. They may be acceptable as a primary basecourse or as an overlay where traffic loadings are less than 500,000 EDA[3]. The production control requirements set out in this report may be relaxed to some degree for the construction of a pavement

1 TNZ = Transit New Zealand

2 M/4 specification = Crushed Basecourse Aggregate (1984)

3 EDA = Equivalent Design Axles

designed for traffic loadings of less than 200,000 EDA, and when a higher degree of risk may be justified.

Three rock types, listed below, are covered by this report but the concepts outlined can also be applied to most non-standard aggregates.

The rock types are:

1. Wanganui shell rock,
2. North Taranaki andesite,
3. Waiouru volcanic grit.

Aggregates manufactured from these rock types do not meet the TNZ M/4 specification requirements for crushing resistance, sand equivalent and particle size distribution. They are usually soft and may degrade considerably during processing and the construction of road pavements. Mechanical interlock between coarse aggregate particles is not regarded as a fundamental requirement for stability of these basecourses, but the internal frictional properties of the fine fraction of the basecourse is a dominant factor for good pavement performance.

Field assessment, test track trials using the Canterbury University Accelerated Pavement Testing Facility (CAPTIF) and geological investigation, clearly show that several non-standard aggregates in the Wanganui and Taranaki regions are acceptable for pavement construction provided that:

- the pavement design is based on subgrade or sub-base bearing values,

- the construction uses appropriate engineering practice and good quality control to provide a high density pavement,

- good drainage is maintained throughout the pavement life,

- second coat seal is applied within three months of applying the first coat seal.

2. CONCEPT OF PARTICLE SIZE DISTRIBUTION AND TALBOTS 'n' VALUES

Non-standard pavement aggregates in the Wanganui and Taranaki regions are derived from a wide variety of rock types. Although showing little visual similarity to each other, most of these aggregates have some physical characteristics which enable a common approach to the assessment of a deposit for use as a pavement construction material.

Particle size distribution (PSD) is used in most specifications to control quarry production. The concept of PSD control in this report is retained but expressed in terms of Talbots 'n' value. Examples of this method of presentation of PSD curves are given in the Appendix.

The 'n' value represents the slope of the PSD curve (at any part of the curve) and is determined from the equation:

$$P = 100 \left(\frac{d}{D}\right)^n$$

where

	P	=	percentage finer than particle size 'd'
	d	=	particle size (mm)
	D	=	maximum particle size (mm)

Research has shown that the PSD of a basecourse with good particle shape, when plotted on logarithmic axes, is well graded for road pavement construction if the plot of the aggregate is represented by a straight line having an 'n' value between 0.35 and 0.55. If the 'n' value is higher than 0.6 the material is deficient in the finer fractions and can be difficult to place and compact. When the 'n' value is less than 0.35, the material has an over-abundance of fines and may become unstable during construction or early in the life of the pavement.

If the 'n' value approximates 0.5, the relationship of the particle sizes are such that high density can be achieved with only adequate compaction. Soft or weak aggregates tend to break down readily during the crushing process to form a "dense" grading, i.e. PSD with an 'n' value of 0.5 or less. Hard or strong rocks do not readily produce fines during crushing and often produce basecourse with a high 'n' value. River gravels and other sedimentary deposits with mixed hard and soft particles will generally produce dense graded aggregate.

The 'n' value concept provides a useful method of controlling PSD since an acceptable envelope can be defined by the specification of two 'n' values. The concept also allows for variability in the maximum size of the aggregate. A maximum particle size can be accepted which is economic to produce and lay.

Deposits such as shell rock, andesite, volcanic debris flows and alluvium are frequently layered, with a variation in the maximum particle size. The grading of a basecourse prepared from these is dependent to some extent on the method used to extract and process the material.

Some basecourse materials become unstable when the degree of saturation exceeds 80%. This situation is usually avoided by ensuring that:

- good drainage is maintained throughout the life of a pavement,
- some degree of control of the quantity and quality of basecourse fines (e.g. plasticity, clay index, etc.) is maintained.

Many aggregates of the Wanganui and Taranaki regions that have been used successfully as basecourses, show a marked change in the slope of the grading curve at a particle size of between 0.6 - 1 mm (Figure A1.1). Effective drainage can be achieved when the 'n' value fraction of the aggregate exceeds 0.5.

As the various aggregates in the Wanganui and Taranaki regions are each derived from such different rock types, rigid specification limits are not appropriate. Instead, the concepts of PSD based on 'n' values can be used with the aid of trial sections and laboratory testing, to assess non-standard sedimentary or volcanic aggregates.

3. TESTING

A flexible testing programme is appropriate for the evaluation of the aggregates from this region particularly in view of the natural variability expected in rock and aggregate deposits. Quality control facilities should be available to undertake the following tests:

1. Water Content - NZS 4407, Test 3.1.
2. Californian Bearing Ratio - NZS 4402, Test 6.1.1 using vibratory compaction; Test 6.1.3 in situ test.
3. Clay Index - NZS 4407, Test 3.5.
4. Sand Equivalent - NZS 4407, Test 3.7.
5. Plasticity Index - NZS 4407, Test 3.4.
6. Particle size distribution - NZS 4407, Tests 3.8.1 and 3.8.2.
7. Crushing resistance - NZS 4407, Test 3.10.

4. APPRAISAL OF A ROCK TYPE AS A POTENTIAL SOURCE OF BASECOURSE

Hard rock types such as greywacke and unweathered volcanic rocks produce basecourses that have generally predictable properties. An aggregate produced with good quarry management will generally meet the requirements of a premium grade material (e.g. as given in TNZ M/4). Stockpiling, transport and the construction processes do not usually cause any major changes in the physical characteristics of such aggregates. While minor degradation may occur it is not regarded as significant when conventional construction techniques are used.

The engineering properties of aggregates manufactured from soft rock types, such as rhyolite, andesite, scoriaceous basalt, limestone and soft sedimentary rock, may change significantly from between the quarry and the completed pavement. The physical characteristics may have been either improved or degraded as a result of processing, stockpiling, etc.

In the preliminary appraisal of a deposit, testing should be used to evaluate the suitability of the material as a basecourse for pavements designed to carry low to medium traffic loadings.

4.1 Shell Rock

4.1.1 General

Shell rock is a term used to describe soft sedimentary rock containing many fragments of shell. These low grade limestones are found in several parts of the Wanganui region and show a wide range in the degree of calcareous cementation, quantity of shell and the size of the terrigenous particles. For example, two quarry operators that were producing shell rock aggregate in 1985 were working geologically different deposits. Other deposits will probably be considered for aggregate supply in the future but a prerequisite to operate any new deposit should be a thorough geological appraisal to ascertain the degree of variability of the deposit and to establish if any relationship exists between various deposits. The identification of a relationship would probably simplify the testing required to maintain control of quality of production and give confidence in the use of the aggregate.

4.1.2 Extraction

These limestone deposits need to be stripped of sand, sandy clay and clay materials that would otherwise contaminate the basecourse. Likewise, non-calcareous sedimentary rock must be completely removed where it overlies, or exists as a lateral extension of, the shell rock deposit.

The rock can be quarried by simply ripping each layer in the deposit followed by crushing with the bulldozer tracks until the maximum particle size is approximately 65 mm.

The aggregate should then be placed in layers in a stockpile so that it can be subsequently mixed if necessary with a loader. These activities should be controlled to ensure that:

(i) maximum size is not exceeded,
(ii) basecourse is well graded.

Simply blading the ripped shell rock into heaps can produce a stockpile which is poorly graded. Layered stockpiles are desirable although excess handling can cause unnecessary degradation.

4.1.3 Sampling

Preliminary testing, carried out on the basis of a random sampling plan, should be undertaken when evaluating a deposit. Random sampling techniques are necessary to determine how variable the deposit is, whether processing of the aggregate is required, and whether the aggregate will degrade during handling and construction. Sampling in the quarry stockpile should be designed to establish whether or not the material is suitable for a specific use.

The target sampling frequency is one sample for each 300 m^3 of shell rock aggregate. Where stockpiles are less than 1000 m^3 in size, the following sampling frequency is considered minimal:

Stockpile less than 100 m^3 - 3 samples,
Stockpile 100 - 1000 m^3 - 1 sample per 250 m^3 with a minimum of three samples.

Each sample should consist of at least five increments taken from random positions in a stockpile. The random positions should be determined by procedures outlined by NZS 3111 or NZS 4407.

Each sample should be not less than 50 kg in mass and should be packaged in containers that will maintain the aggregate close to its field water content.

4.1.4 Aggregate Selection Plan

As Wanganui shell rock is a variable product, strict adherence to specified limits may result in rejection of an otherwise economic source of aggregate. The selection of such a material should be planned on the basis of a sequential acceptance procedure. Aggregate selection and control testing use a range of test methods, some of which will be emphasised in this report more than others. A single test result is of little value and decisions should be based on the average value of at least three samples.

4.1.5 Preliminary Design Testing

Preliminary design testing of a deposit should involve at least three samples selected from the quarry face or from a stockpile, representing the range of lithologies or rock types in the deposit. The shell rock should be crushed or broken up so that the maximum size of the aggregate does not exceed 65 mm.

The performance of shell rock in a pavement is a function of the strength of the aggregate system and the frictional characteristics of the sand/silt/clay fractions. Testing to establish such properties involves the Californian Bearing Ratio (CBR) and clay index (CI) tests, and the particle size distribution (PSD) of coarse and fine aggregate fractions. The values that should be obtained from these tests are discussed below, and the sequence of tests is outlined in Figure 1.

Californian Bearing Ratio (CBR). The strength of vibrator-compacted shell rock, as determined by the CBR test (Section 3.0) after 4 days soaking, should be not less than 120%[4].

Clay Index (CI). The potential of shell rock to develop plasticity from the clay minerals contained within the fabric of the rock or aggregate, or from contamination of the quarried rock, should be assessed using the Clay Index test (Section 3.0):

(i) CI of crushed shell rock powder should be less than 5,
(ii) CI of basecourse fines should be less than 5.

When the clay index value exceeds 5, further testing for plasticity is required using the Atterberg Limit Tests.

Atterberg Limit Tests (ALT). Plasticity Index (PI) should be less than 5 for aggregate to be suitable. If PI is more than 5, aggregate may be suitable for use as sub-base.

Particle Size Distribution (PSD). The stability of the compacted shell rock is largely dependent on the 'n' value of the aggregate 'fines'. This is defined as the aggregate fraction either coarser or finer than 1 mm as determined from a log/log plot of the PSD. Typical results for shell rock aggregate show that the PSD graphs will generally consist of two lines intersecting at about a particle size of 1 mm. Further discussion is given in the Appendix.

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4 CBR values greater than 200% are frequently obtained from shell rock. CBR values greater than 120% may be required where traffic loadings are higher than 5×10^5 EDA or where aggregate is to be used as an overlay.

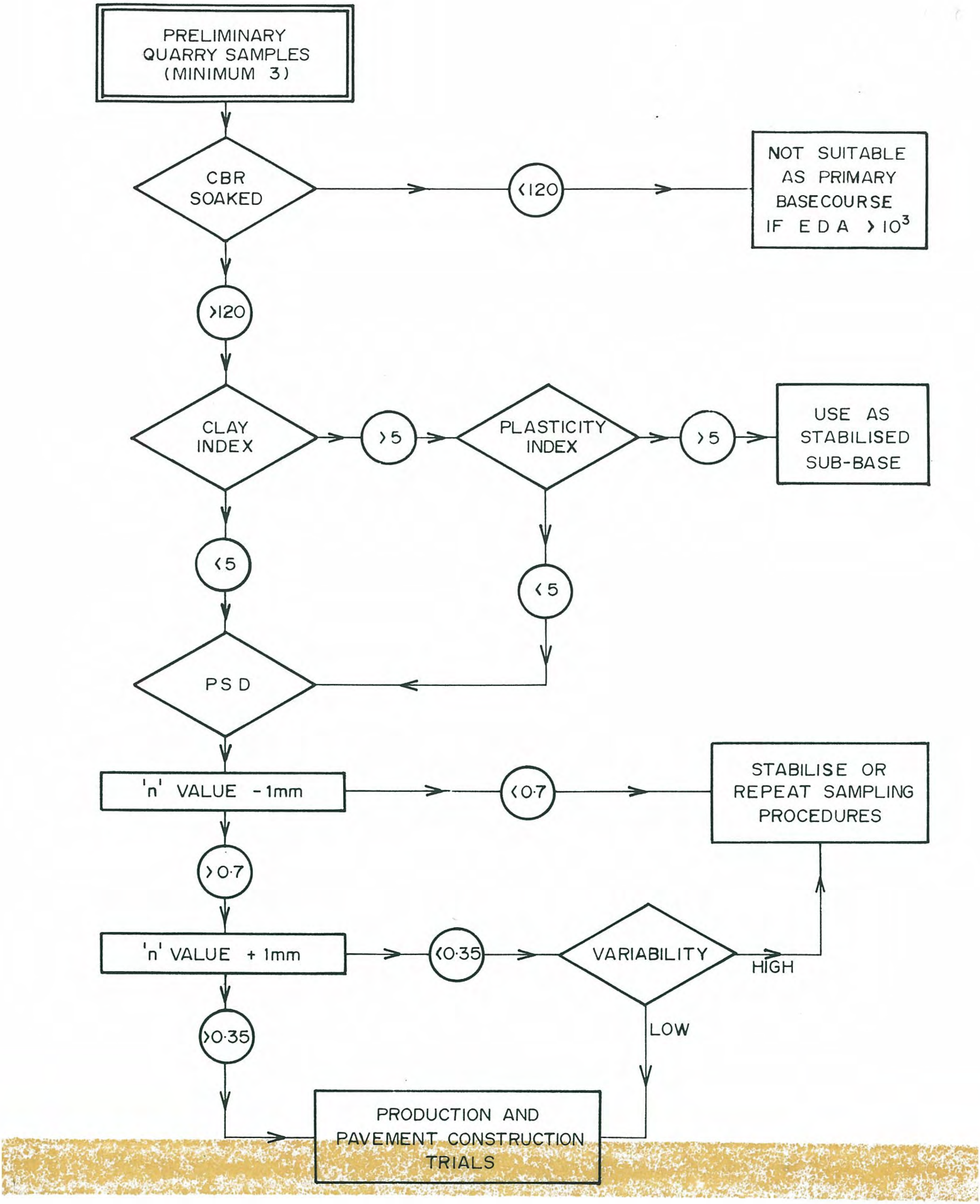

Figure 1. Evaluation chart for Wanganui shell rock aggregates.

The PSD should be determined in accordance with NZS 4407 (Section 3.0).

For stability and strength in a pavement constructed of shell rock aggregate, the following criteria are considered to be minimum values, with greater emphasis being placed on the 'n' value of the finer fractions:

(i) 'n' value of fines determined, using a wet-sieving process, should be not less than 0.70,

(ii) 'n' value of material coarser than 1 mm, using a dry-sieving process, should be greater than 0.35.

Crushing Resistance. The test is not considered essential to evaluate a deposit of shell rock. However, a value not less than 50 kN is considered to be adequate and should ensure that crushing under the construction machinery will not be excessive.

4.1.6 Tests for Final Design and Contract Acceptance

Aggregate sources which satisfy the requirements outlined in Figure 1 should then be evaluated by a production and construction testing programme. Marginal or variable results may indicate that the shell rock may be not suitable, or have to be treated with a stabilising agent. A broader based testing programme using Atterberg Limit tests may be needed if CI tests give marginal results. Additional sampling and testing is essential if the results from two samples show marked variation.

4.2 Andesite

4.2.1 General

Basecourse aggregates from the Taranaki and Waiouru regions are produced from a wide range of deposits. Andesite rock is quarried and crushed from material taken from pits in alluvial and volcanic debris flow deposits on the flanks of Mt Taranaki and Mt Ruapehu. Many of the smaller pits are operated intermittently but some in the New Plymouth and Stratford areas are operated continuously. The alluvial deposits are highly variable and the control of quality will always be a problem. The strength of many rock particles is well below the hardness required by the TNZ M/4 specification and weathered rock may be present in significant quantities.

A prerequisite to working a deposit is a geological appraisal to ascertain the degree of variability of the rock. Many deposits are layered and blending of more than one of the layers may be desirable to produce a comparatively consistent aggregate. The term "pit won andesite" is a convenient one to cover the aggregates produced from various alluvial and/or debris flow deposits in this area.

4.2.2 Extraction

As these aggregate deposits are variable, methods of extracting rock from them for basecourse production will depend on the characteristics of the deposit. Extraction methods should be aimed at reducing variability in the final product as much as possible, and special equipment or procedures may be necessary to ensure that the basecourse is well mixed to ensure uniformity.

4.2.3 Sampling

The procedure adopted for sampling will depend on the type of deposit being worked. Open cut excavation or test pits enable a visual assessment of the variability to be made and present opportunities to select samples of rock fragments for hardness testing or clay layers for CI testing. Samples of the excavated material can be crushed and blended so that the characteristics of the mixed aggregate can be determined. Random sampling is necessary and a sampling plan should be prepared. Many of these deposits will contain rock which is relatively soft and the results of crushing tests may often fall below the requirements of TNZ M/4 specification.

The target sampling frequency is one sample for each 300 m^3 of aggregate. Where stockpiles are less than 1000 m^3 in size the following sampling frequency is considered minimal:

Stockpile less than 100 m^3 - 3 samples,
Stockpile 100 - 1000 m^3 - 1 sample per 250 m^3 with a minimum of three samples.

Each sample should consist of at least five increments taken from random positions in a stockpile. The random positions should be determined by procedures outlined by NZS 3111 or NZS 4407.

Each sample should be no less than 50 kg in mass and should be packaged in containers that will maintain the aggregate close to its field water content.

4.2.4 Aggregate Selection Plan

As andesite aggregates are so variable, strict adherence to specified limits may result in rejection of an otherwise economic source of material. The selection plan should involve a sequential acceptance procedure which will be dependent on the degree of crushing required. Pit gravels which include all the excavated material, even that with minimal crushing resistance, may be included as potential material for basecourse. Aggregate selection and control testing uses a range of test methods, some of which will be more appropriate than others. Additional sampling is needed when the results from two samples show differences, and decisions should be based on the average value of at least three samples.

4.2.5 Preliminary Design Testing

Preliminary design testing of a deposit should include testing of at least three samples selected from the pit face or stockpile to allow for the range of lithologies of the deposit. The tests and the values that should be obtained to meet design standards are discussed below, in order of the sequence of tests outlined in Figure 2.

Crushing Resistance. This test is considered desirable when evaluating a deposit. A minimum value of 100 kN at the time of crushing is considered adequate, but particle shape has a considerable influence on this value. An increase in the percentage of rounded or water-worn particles will increase the value, whereas crushing that produces flakey and/or elongated particles will result in a lower value. Degradation under construction equipment will also increase the crushing resistance.

Sand Equivalent (SE). Many deposits of andesitic gravel contain volcanic ash of silt or clay size which can block pore spaces in the basecourse and reduce the permeability markedly. The most satisfactory method of controlling this percentage of fine material is to ensure that the SE value is above 40.

If the SE is consistently below 40 the fines of the basecourse should be checked for clay minerals using the CI test.

Clay Index (CI). This test should be used to check the fines of the basecourse if the SE is consistently below 40. The CI value should be less than 4.

Particle Size Distribution (PSD). The stability of the compacted andesite aggregate is largely dependent on the high frictional characteristics of the particles of volcanic rock. Excessive silt-sized material, often present in alluvial deposits, can result in saturation of a compacted basecourse in wet conditions.

The minimum 'n' values required to ensure adequate stability and strength are given below. Greater importance is placed on the 'n' value of the material finer than 1 mm (see Appendix):

(i) 'n' value of fines (i.e. <1 mm), using a wet-sieving process, should be not less than 0.70,

(ii) 'n' value of coarse material (i.e. >1 mm), using a dry-sieving process, should be greater than 0.45.

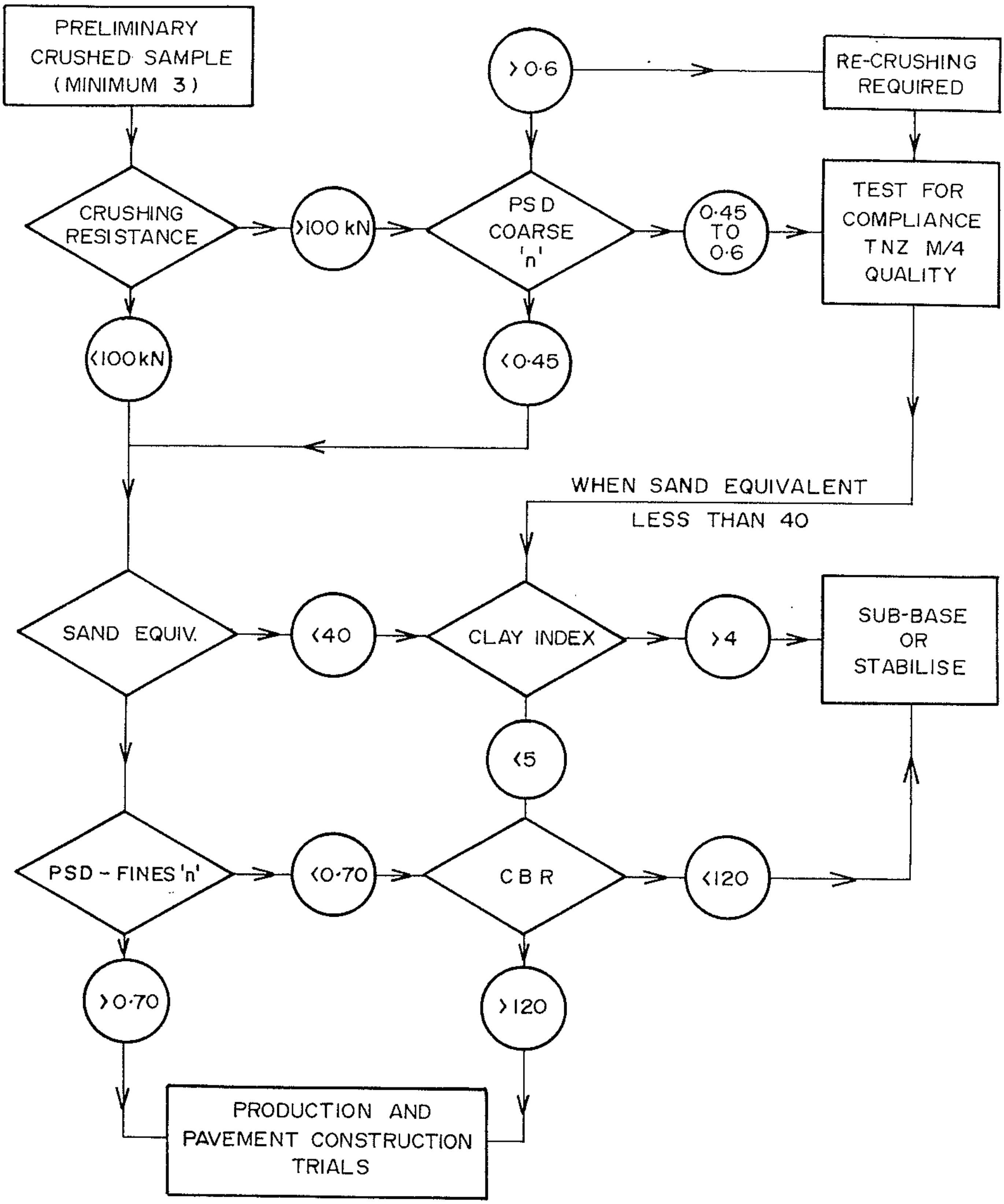

Figure 2. Evaluation chart for Taranaki andesite aggregates.

Californian Bearing Ratio (CBR). If the crushing resistance value is less than 100 kN the strength of the compacted aggregate should be determined using the CBR test carried out after four days soaking. The test value obtained should be greater than 120%.

4.2.6 Tests for Final Design and Contract Acceptance

Aggregates which satisfy the above requirements (Figure 2) should be suitable for basecourse on roads carrying low to medium traffic densities. If results of the tests are highly variable, additional sampling may be necessary to establish whether stabilisation with a chemical additive is necessary. Production and pavement construction trials are desirable to assess changes in the properties under normal stockpiling, placing and construction techniques.

5. AGGREGATE PRODUCTION

5.1 Production Control

The stability of a basecourse as a load-bearing pavement material is a function of many factors including those relating to the service environment, traffic loadings, subgrade strength, as well as construction methods and material quality.

The objective of production control is to maintain quality of aggregate for use in pavement construction. When the properties depart from those of a premium quality material, the effect of each specific characteristic needs to be assessed in relation to the production process and the construction methods. Aggregates from the Wanganui and Taranaki regions are softer than premium grade rock and some have SE below the TNZ M/4 requirements of 40. They may be used for pavements but they require product and construction control and techniques different to those for an TNZ M/4 material. The sequence for testing such aggregates is outlined in Figure 3.

5.2 Shell Rock Aggregates

Preliminary testing of an aggregate from a pit to prove that it will give adequate performance, involves tests which are time consuming and not readily applicable to production control testing. CBR values, CI and 'n' values are still important and appropriate tests should be undertaken during the construction stage but additional tests need to be introduced as supplementary quality control tools.

5.2.1 Sampling

Aggregate testing for production control purposes is best done on two fractions separated by sieving samples of the freshly crushed rock:

(i) Coarse aggregate - retained on the 4.75 mm sieve,
(ii) Fine aggregate - passing 4.75 mm sieve.

The samples should be stored in containers, at their field-water content, until tested. Results from three samples, which is the minimum number of samples for field trials, should be averaged and not treated as individual samples.

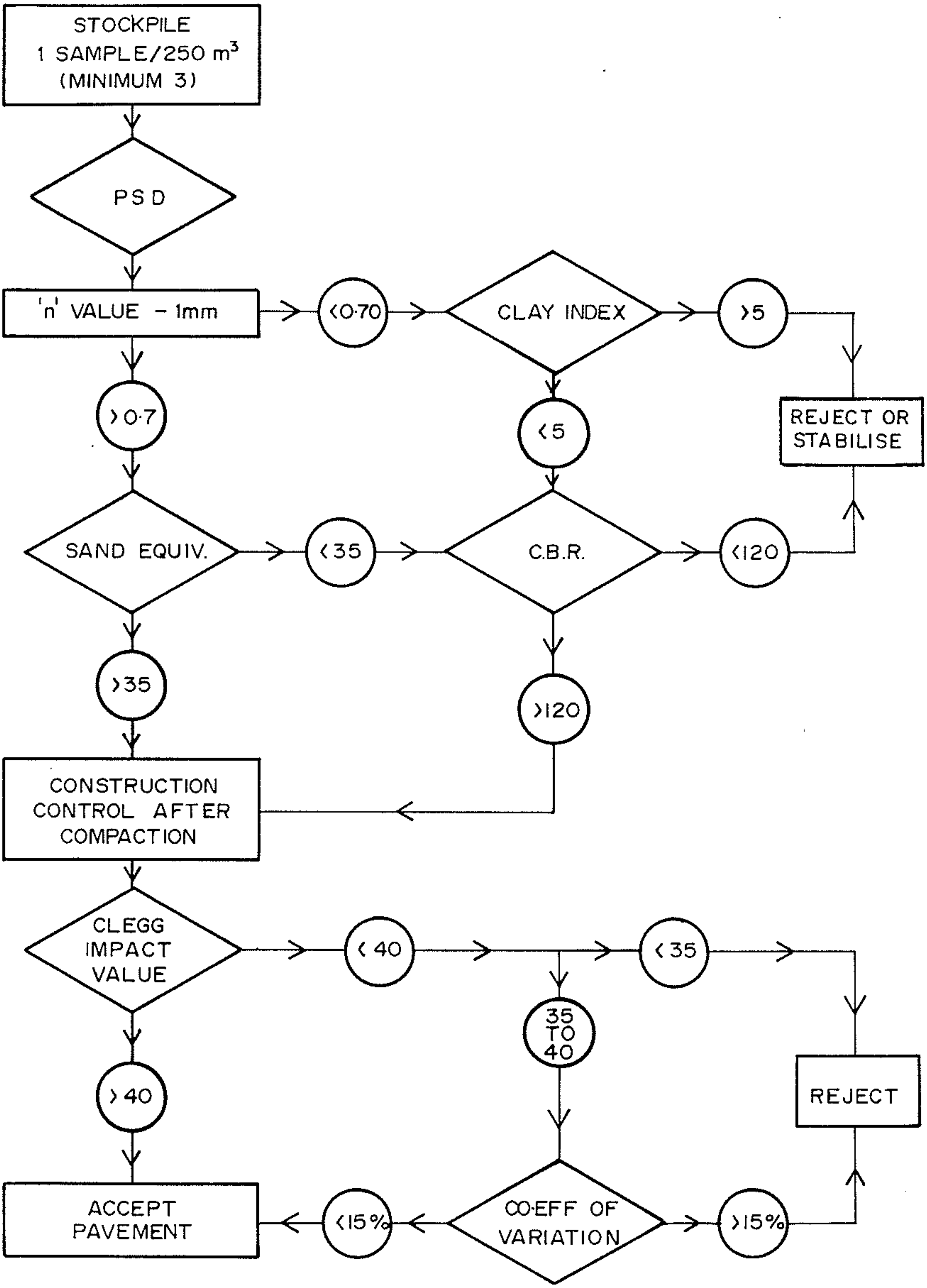

Figure 3. Selection chart for unbound basecourse from Wanganui shell rock or Taranaki andesite.

5.2.2 Particle Size Distribution

The test methods for determining PSD must follow NZS 4407 Test 3.8. Dry sieving of coarse aggregate is permitted but fine aggregate should be wet-sieved. However, if the percentage passing the 0.075 mm sieve is less than 2% in the initial appraisal of the material, dry sieving can be used.

The results obtained should be plotted on a log/log graph and a best fit straight line should be drawn between:

(i) Maximum particle size and 4.75 mm sieve,
(ii) 0.6 mm and 0.075 mm sieves.

The gradient of the straight line graphs should have 'n' values not less than 0.35 and 0.7 for (i) and (ii) respectively and should intersect at about the 1 mm particle size.

5.2.3 Sand Equivalent

The SE determined on the fine aggregate fraction can be used as a rapid field test to evaluate the dirtiness of the shell rock and, to a limited degree, the 'n' value. Both these factors may vary depending on the composition of the deposit. Excessive breakdown caused by handling, stockpiling or weathering may lower the SE and increase the CI.

The SE value should be greater than 35. Values lower than this indicate contamination from overburden or an increase in the clay content caused by changes in the composition of the rock or the deposit. A coefficient of variation not greater than 20% is recommended to ensure production of an aggregate of consistent quality.

5.2.4 Clay Index

Although the test is not essential for production control unless the SE value is consistently less than 40, the CI should not exceed 5.

5.2.5 CBR Values

Samples compacted in the laboratory, and then soaked for four days, should have a CBR value in excess of 120%. The necessity to carry out the CBR test will be apparent if:

(i) 'n' values are borderline,
(ii) SE is always below 40,
(iii) CI is marginal.

If the CBR value on a compacted sample is less than 120% the material should be rejected, or stabilisation using chemicals should be investigated.

5.3 Andesite Aggregates

Testing for production control of andesite aggregates should use the SE and PSD of the fine fraction. The more time-consuming tests such as CBR and CI should be included when the results of PSD and SE are marginal.

5.3.1 Sampling

Aggregate testing for production control purposes is best carried out on two fractions separated by sieving samples of the freshly crushed rock:

(i) Coarse aggregate - retained on the 4.75 mm sieve,
(ii) Fine aggregate - passing 4.75 mm sieve.

The samples should be stored in containers, at their field-water content, until tested. Results from three samples should be averaged and not treated as individual samples.

5.3.2 Particle Size Distribution

The test methods for determining PSD should follow NZS 4407 Test 3.8. Dry sieving of coarse aggregate is permitted but fine aggregate should be wet sieved. If the percentage passing the 0.075 mm sieve is less than 2%, dry sieving of both fractions can be permitted.

5.3.3 Sand Equivalent

The SE determined on the fine aggregate fraction can be used as a rapid field test to evaluate the dirtiness of the andesite aggregate and, to a limited degree, the 'n' value. A change in dirtiness or in 'n' value of fines can indicate that marked changes have occurred in the composition of the deposit. Excessive breakdown caused by handling, stockpiling or weathering may lower the SE.

The SE test value should be greater than 35. Values lower than this indicate contamination with volcanic ash. A coefficient of variation not greater than 20% is recommended to ensure the production of an aggregate of consistent quality.

5.3.4 Clay Index

This test is not essential for production control unless the SE value is less than 40. When the SE value is below 40 the CI shall not exceed 5.

5.3.5 CBR Values

Samples compacted in the laboratory, and then soaked for four days, should have a CBR value in excess of 120%. The necessity to carry out the CBR test will be apparent if:

(i) 'n' values are borderline,
(ii) SE is always below 40,
(iii) CI value is marginal.

If the CBR value on a compacted sample is less than 120% the material should be rejected, or stabilisation using chemicals should be investigated.

5.4 Test Sections

Test sections are essential when assessing non-standard aggregates for use as basecourse. Soft rocks, such as shell rock, will degrade and changes can be expected to occur in the PSD and SE during construction. Experienced gained during the construction of a test section and from the monitoring of its performance should provide the practitioner with more knowledge and confidence in the use of the material.

Refer to RRU (1979) Technical Recommendation TR3, "Comparative Pavement Trials for New Zealand Conditions", for guidance in the construction and monitoring of pavement test sections.

6. BASECOURSE CONSTRUCTION

6.1 Construction Control

The performance of a pavement layer depends, to a large degree, on the characteristics of the foundation on which it has been placed. It is essential that the condition of the subgrade is fully investigated particularly before a pavement using aggregates of marginal quality is constructed. Attention must also be paid to ensure uniformity of strength of the subgrade and the provision of adequate cross-fall and drainage.

As stated in Section 5.4, test sections are an important part of the process for selecting non-standard aggregates for road construction. They should be constructed with aggregate as similar as possible to that to be used in the final roading project. Experimentation with pit-run or crushed rock can assist to establish the most economic means of construction and most practical methods of control testing. Aggregates prone to degradation should be compacted to high density at a water content close to optimum. Measurement of density of the compacted aggregate by nuclear methods will provide good control as will the CBR test. Use of the Clegg Hammer test instead of the CBR test is discussed below.

6.1.1 CBR Values

CBR can be measured in accordance with NZS 4402 but it is an expensive, time-consuming operation. The more appropriate test method uses the Clegg Impact Hammer which can be used to test the uniformity of a compacted pavement as well as to provide an equivalent CBR value.

Correlations shows that a Clegg Impact Value (CIV) of 40 is equivalent to an in situ CBR value of 120%. CIV of 40 and above is an acceptable target average for a minimum of ten CIV tests taken in random positions over any 50 m length of compacted pavement.

A lower value of 35 can be accepted, provided the standard deviation of the ten tests is not greater than five and the coefficient of variation is less than 15%.

If the pavement does not comply with the above criteria one of the following procedures may be necessary:

(i) Carry out CBR tests in accordance with NZS 4402 Test 6.1.3,
(ii) Carry out further compaction of the pavement and re-test,
(iii) Test compacted aggregate for PSD.

6.1.2 Particle Size Distribution

The 'n' value of the fine fraction should not be less than 0.60.
The coarse fraction should have an 'n' value not greater than 0.60.

6.1.3 Clay Index

CI test is not normally carried out after an aggregate stockpile has been accepted as suitable, but it may be necessary if unsatisfactory bearing values have been obtained. Only values less than 5 should be accepted. The need to control CI generally applies to shell rock but may also be applicable to volcanic aggregates produced from alluvium derived from volcanic deposits, in which clay may be a contaminant.

6.1.4 Sand Equivalent

The SE test may be made on damp or dry sieved samples for rapid field evaluation of possible contamination from clay or excessive degradation to clay. A target value of 35 is recommended for shell rock and andesite aggregate basecourse. Many andesite aggregates produce a basecourse with an SE value greater than 40 after compaction. This value indicates that the products of degradation are sand, not clay, and are not detrimental to the compacted pavement.

6.1.5 Aggregate Rejection

If the compacted aggregate has an average CIV less than 35 and fails to meet the requirements of PSD, CI and SE (Sections 6.1.2, 6.1.3, 6.1.4 respectively), its suitability for use as a basecourse should be re-evaluated. It should then be tested for its reactivity to lime or cement to determine if it can be stabilised by either or both of these materials. It should not be accepted as suitable for use as a primary base layer.

6.2 Construction Techniques

Experience has shown that aggregate manufactured from soft rock cannot be successfully used unless special precautions are taken during construction. Aggregate must be delivered to the road bed from a stockpile which has been built by layering. The PSD of aggregates from stockpiles which have been formed by end dumping from trucks, or taken from storage bins, will be variable and the aggregate should not be used.

Shell rock needs to be compacted at its natural water content or slightly above its optimum water content, and must not be permitted to dry out during the compaction process. Final pavement levels are best achieved by cutting to those levels rather than filling depressions with thin layers of fines. When 'n' values are below 0.5, further changes in PSD will be minimal, and a high density should be achieved with average energy input by rollers. Stability of such aggregates is achieved by compacting the aggregate to a high density (low voids) condition so that it becomes virtually impervious.

Andesite aggregate can be laid in accordance with standard procedures but care is necessary with the application of water during rolling because many andesitic rock types are porous and absorb water.

It is also important to maintain the water content of the basecourse approximately at its optimum water content. Both shell rock and andesite basecourses will resist compaction when drier than optimum but will become unstable at high water contents. Compaction at the optimum water content should enable a density of not less than 100% of the laboratory standard density to be achieved.

The type of road roller to use on shell rock or andesite basecourses is best determined by experimentation since it depends on the composition and hardness of the coarse particles by particle shape, sorting of the coarse fraction, porosity and inter-particle friction. Rollers that have been used successfully with these materials are grid-steel-wheeled vibrating and pneumatic-tyred rollers. Short test sections will be of considerable benefit for establishing suitable construction methods and equipment.

Most soft aggregates will degrade so that the finished surface of a well compacted pavement will often be coated with a layer of fines. If steel-wheeled rollers and normal fine-spray watering techniques have been used during construction, the final surface may be coated with a slurry. However, a last fine grader cut, an application of a single sized hard chip or gravel as a running course, and normal traffic use should provide a surface satisfactory for sealing. This procedure is most applicable when shell rock aggregate is used.

Experience has shown that a first coat seal applied to a very dense surface can break down, particularly under wet conditions. Application of the second coat seal as soon as possible after the first coat seal is important to ensure stability in the protective surfacing. Experimental trials utilising a technique proposed by Brenan (1987) may be warranted. This technique involves "locking" a first coat seal of 12 mm chip with a 4 mm chip overlay that is rolled in.

Samples of compacted aggregates should meet the requirements for PSD, SE and CI tests given in Section 5. In situ CBR tests on the test sections are useful but may be replaced with CIV (see Section 6.2).

7. SUMMARY

Experience has shown that aggregates obtained from rock sources in the Wanganui and Taranaki regions can be used for pavements which have design loadings of less than 500,000 EDAs. These aggregates, although not complying with TNZ M/4 specification, need to be of a consistent quality to limit the risk of failure. Specifications adapted to deal with the wide range of aggregates available are required to control the manufacture of the basecourse and method of construction.

"Pit run", as distinct from "pit won", aggregate is not recommended as basecourse for pavements designed for such loads unless consistency within it can be proven. It has been found that some level of processing is required in order to produce a basecourse of appropriate quality. However, unprocessed material may be used successfully as a sub-base.

Controlling the PSD, using the 'n' value concept, enables the best use of a deposit which has a definite variation in maximum particle size. The properties of these aggregates for construction are also best assessed using 'n' values rather than maximum particle sizes.

The CI Test is important in the preliminary evaluation of a deposit but is not considered to be necessary as a control test.

The SE Test also provides a measure of PSD and is a tool to measure consistency of the fraction finer than 5 mm.

Although minimum values for CI and SE have been suggested, the consistency aspect is stressed by recommending a coefficient of variation of less than 20%.

Strength under saturated conditions is the most important criterion in aggregate selection. The performance of aggregates which are prone to degrade during construction can be more readily evaluated by using the Clegg Impact Hammer rather than the CBR test.

Degradation of these aggregates will produce excess surface fines after rolling, watering and shaping, and therefore special techniques are required to ensure that premature failure of the seal coats do not occur. For example the second coat should be applied as soon as possible after the first coat seal. It is essential that these pavements are covered with a durable water-proofing membrane, otherwise water, seeping through the seal, will soften the aggregate layer, potholes will form and the pavement may unravel. Seal failure in wheel paths is common, particularly where cross-falls are inadequate.

In the Wanganui and Taranaki regions, aggregates with variability may be acceptable for pavement with design loadings lower than 200,000 EDA. Application of the techniques described in this report, the use of test sections and good engineering practice should provide the confidence necessary to ensure adequate performance of these aggregates.

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APPENDIX

THE ROLE OF TALBOTS 'n' VALUE IN PARTICLE SIZE DISTRIBUTION

Particle size distribution (PSD) of an aggregate is normally specified by maximum and minimum values of particle sizes for a range of sieve sizes. The concept of using the 'n' value as a guide to aggregate quality and control is a very minor extension of the PSD test but can provide very useful information particularly if the maximum particle size is liable to change, as it may with non-standard or soft aggregates.

Basically the 'n' value determines the slope of a line joining the maximum particle size to the point defined by the smallest size and the percentage passing that sieve when plotted on logarithmic axes. It is determined from the following equations:

$$P = 100\left(\frac{d}{D}\right)^n \quad \text{or} \quad n = \frac{\log P - 2}{\log d - \log D}$$

where P = percentage finer than particle size d,
d = particle size (mm),
D = maximum particle size (mm).

Figure A1.1 shows the particle size distribution of three basecourse aggregates. The graph for shell rock aggregate consists of two relatively straight sections whereas that for andesite basecourse, although it is of similar shape, deviates significantly from a straight line. The grading of Auckland basalt is virtually a straight line and represents the PSD of a soft rock after compaction. The crushed greywacke river gravel, a hard rock aggregate, sags well below the straight line. The 'n' values, represented by the dotted lines are calculated using the maximum sieve size (37.5 mm) and the smallest sieve size (0.075 mm). The respective 'n' values for the three materials, expressed to the nearest 0.05, are n = 0.45, n = 0.5 and n = 0.55. The compaction characteristics of andesite and greywacke basecourses are quite different.

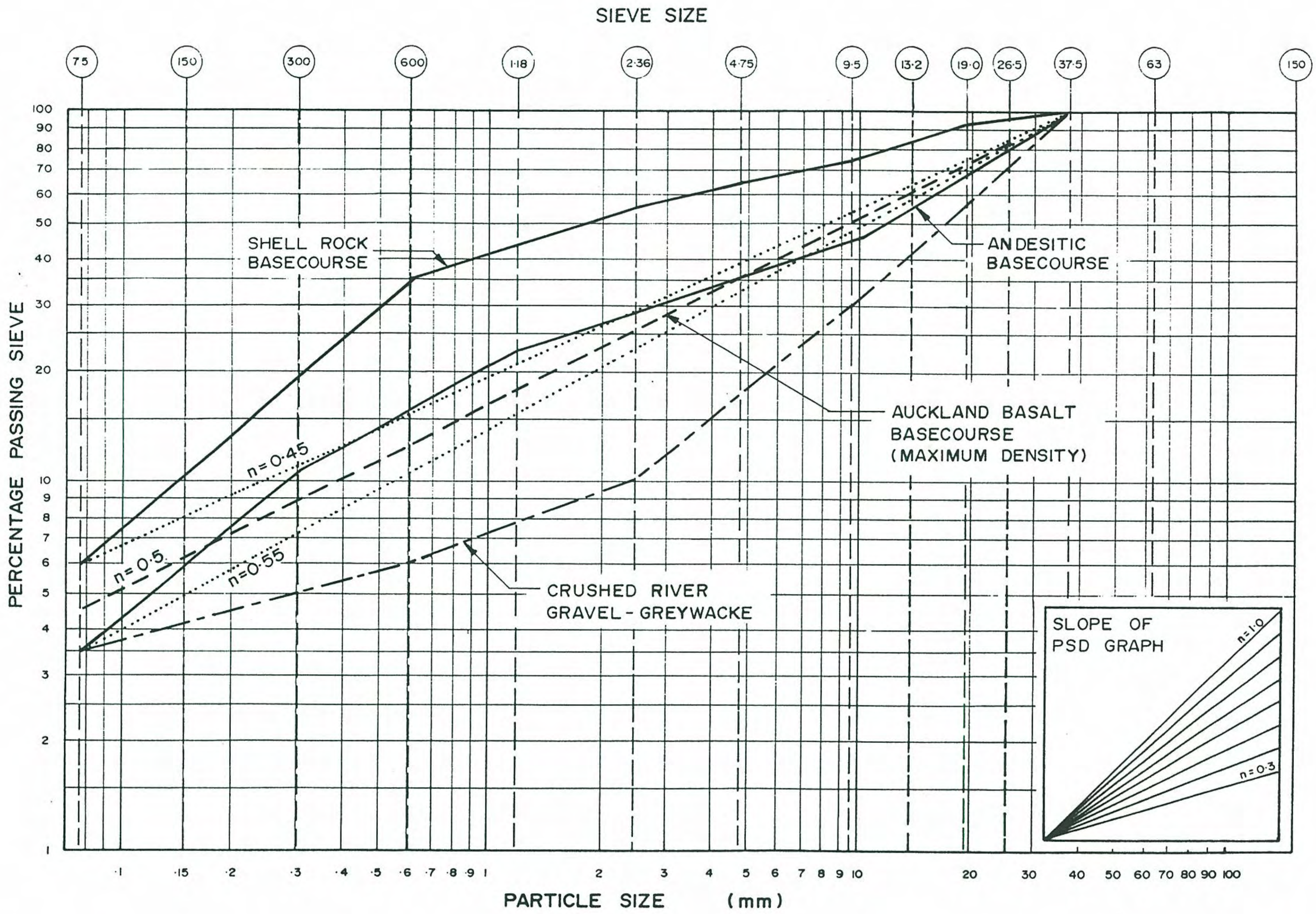
Another method of presentation is shown in Figure A1.2, where the ratio d/D is plotted against P. The shapes of the graphs are similar to those shown on Figure A1.1 although the 'n' values have been calculated for the two sections of the PSD rather than for the overall range shown in Figure A1.1. Recognition of the slope change in a grading curve is important irrespective of whether P is plotted against d/D or the particle size. Use of the d/D plot enables direct comparison of grading irrespective of maximum particle size (see Table A1).

An aggregate 'n' value less than 0.5 could be expected to have good compaction characteristics and should form a dense pavement layer. If the 'n' value of the fine fraction is greater than 0.7 after compaction, the volume of pore spaces present will ensure a relatively high permeability, i.e. good drainage. The change in slope of the PSD graphs is therefore an important factor in assessing aggregates for pavement use.

The concept of 'n' values is particularly useful for the control of pit-run aggregates which may vary in maximum size. Figure A1.3 shows a series of PSD graphs having a constant 'n' value but a change in maximum particle size. To obtain the same placing and compaction characteristics a pit-run aggregate with a minimum size of 65 mm requires less fines than an aggregate with a maximum size of 9.5 mm.

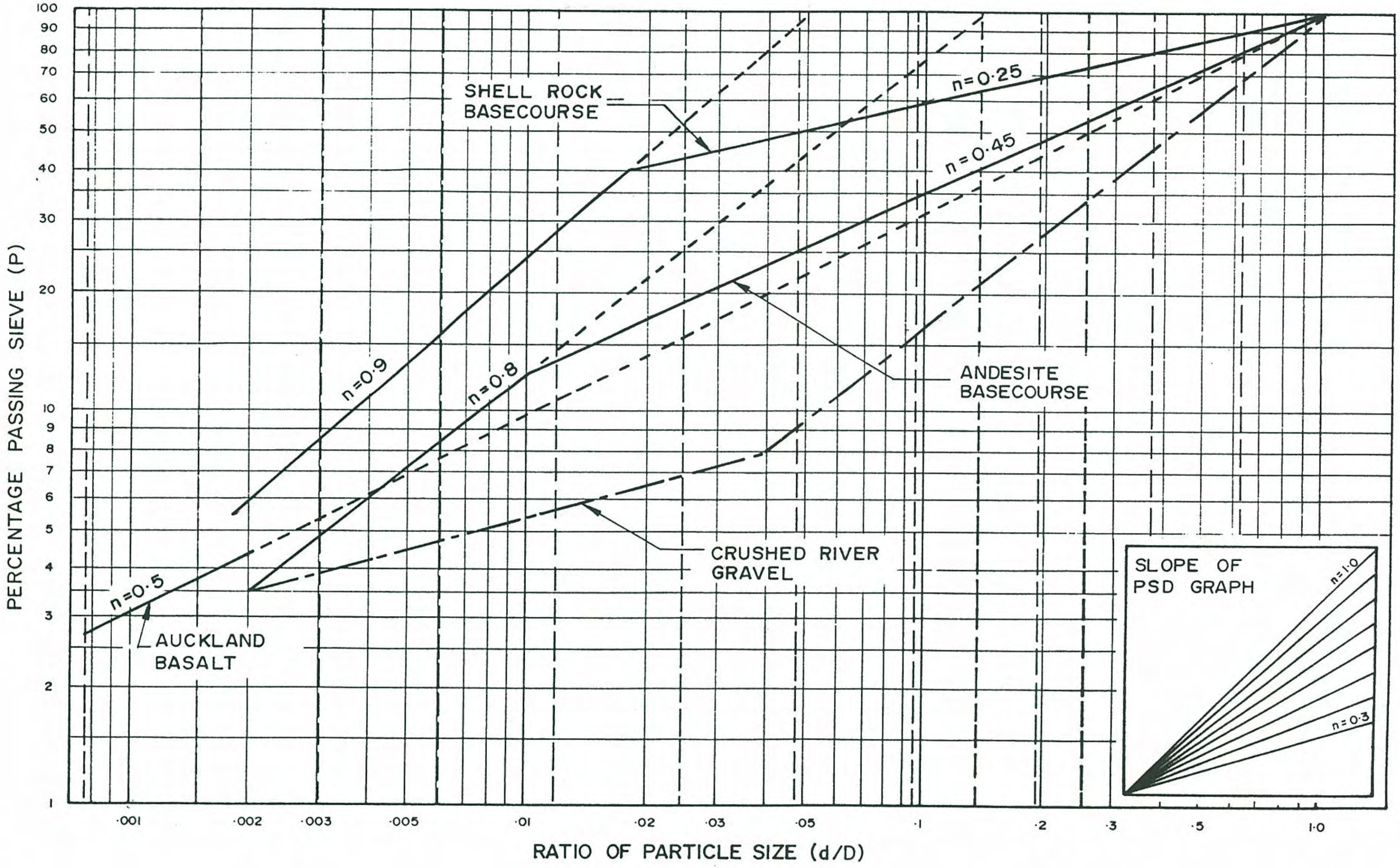
Table A1 d/D Values for Standard Aggregate Sizes

	Maximum Particle Size (D)		
Sieve Size (d) (mm)	19 mm	37.5 mm	63 mm
0.075	.004	.002	.001
0.150	.008	.004	.002
0.300	.016	.008	.005
0.600	.032	.016	.010
1.18	.062	.031	.09
2.6	.14	.069	.04
4.75	.25	.127	.075
9.5	.5	.12	.15
19.0	1.0	.51	.3
37.5		1.0	.6
63.0			1.0



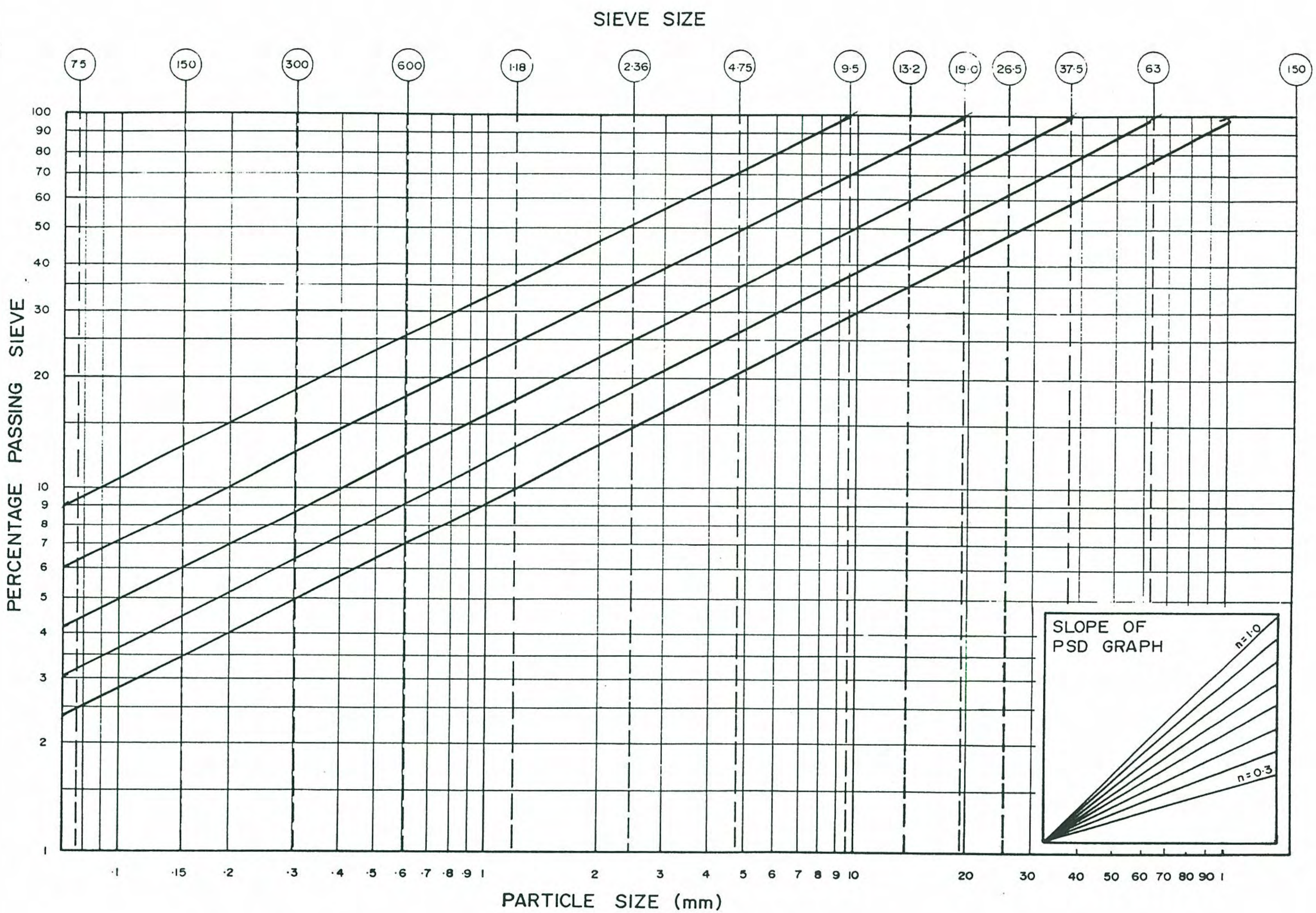
PARTICLE SIZE DISTRIBUTION GRAPH

FIG. A1·1



PARTICLE SIZE DISTRIBUTION GRAPH
OF FIG. A1·1 PLOTTED AS d/D RATIO.

FIG. A1·2



PARALLEL P.S.D. GRAPHS
FOR 'n' = 0·5

FIG. A1·3