# Notes for Design and Indirect Tensile Strength Testing of Modified and Bound Pavement Materials

## 1 PREAMBLE

These notes are for the guidance of the laboratory staff who carry out mix design and quality assurance testing of modified and bound pavement materials, and for the guidance of supervising officers and design engineers commissioned to draft tender documentation and must not be included in contract documents.

## 2 SCOPE

The purpose of T19 is to establish a framework of consistent laboratory based procedures for the preparation of the aggregate, the manufacture and the testing of samples that have been mixed with cement, lime, foamed bitumen and/or bitumen emulsion to ensure that the pavement design engineer receives mix design results that are based on a consistent procedure. The intention is to determine the optimum type and amount of binder(s) being mixed with the pavement materials and/or other aggregates that will achieve the pavement design parameters.

The mix design is the process of identifying and quantifying the mechanical properties of an aggregate/binder mixture in relationship to type and dosage of binder under selected conditions. The "optimum" binder content will usually be the percentage of binder at which the minimum required mechanical design properties of the material are met. It should however be noted that a minimum binder amount is required for purposes of durability. Guidance to the minimum amounts of binder(s) is given in section 9.3 of these Notes.

A stabilisation mix design is a process rather than a procedure in that it involves a sequence of stages where, based on the outcomes of the previous stage, a choice of binder type(s) and/or binder content range(s) has to be made for the following stage rather than a start to finish series of tests. Some notes that relate to the overall procedure from the initial sampling to the final evaluation of the results are provided specifically for the design engineer.

The design engineer and the laboratory staff must interact throughout the whole process because most of the choices that are made aim at simulating the actual stabilisation operation, which will depend on design assumptions, project specifications and/or construction requirements/limitations, which may be unknown to the laboratory staff.

## 3 AGGREGATE PREPARATION AND TESTING

### 3.1 Sampling and Blending of Aggregates

The design engineer is required to give instructions in terms of the location(s) and depth(s) of test pits from which the sample will be retrieved and utilised for mix design testing. It is anticipated that the test pit logs will be reviewed along with any geometric considerations such that the design engineer can determine the appropriate depth and proportion of materials to be prepared for testing. Refer to AGPT, Part 5, Appendix B.5 for guidance. Refer to section 9.4 of this document for guidance on blending of materials.

### 3.2 Aggregate Characterisation

Sections 9.2 and 9.3 of these notes gives guidance for the design engineer to the characterisation of sampled materials and binder selection. Typically, sufficient particle size distribution and plasticity index testing will be undertaken to categorise the suitability of materials for treatment.

### 3.3 Adjusting Aggregate Particle Size Distribution (if required)

The strength of stabilised aggregates largely depends on the particle size distribution (grading). If there is a deficiency in the grading, it should always be addressed by blending in an imported suitable aggregate rather than by increasing the binder content. However, this can be a costly operation. Basecourse and sub-base materials in New Zealand are generally well graded and hydraulic binders are sufficiently forgiving to allow working the material as is. Therefore correction of the grading on the test pit samples is not strictly necessary for cement/lime stabilisation mix designs.

For foamed bitumen, the grading should always be checked first. The relationship in Equation 1 below can be used to design the grading to minimise the air voids in the compacted aggregate for a given maximum particle size and filler (fines) content:

$$P = (100 - F).\frac{d^n - 0.075^n}{D^n - 0.075^n} + F \qquad \text{Equation 1}$$

Where d = selected sieve size (mm)
P = percentage by mass passing a sieve of size d (mm)
D = maximum aggregate size (mm)
F = percentage fines content (inert rock fines plus active binder)
n= variable dependant on aggregate packing characteristics (for foamed bitumen n = 0.45 has generally been adopted)

It is useful to plot the square root of the sieve size on a normal scale as per Figure 1 below. A straight line on this plot represents an ideal particle distribution for foamed bitumen stabilisation.

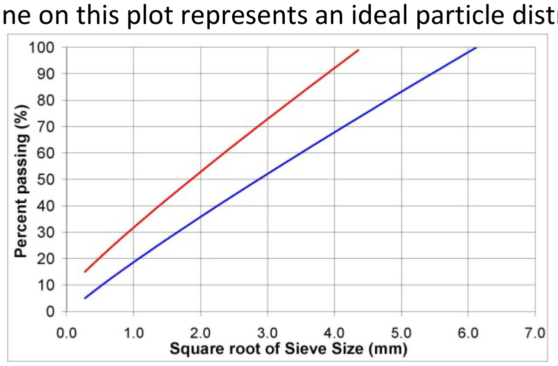



Figure 1: Ideal grading envelope for Foamed Bitumen treated material using the square root of the sieve sizes

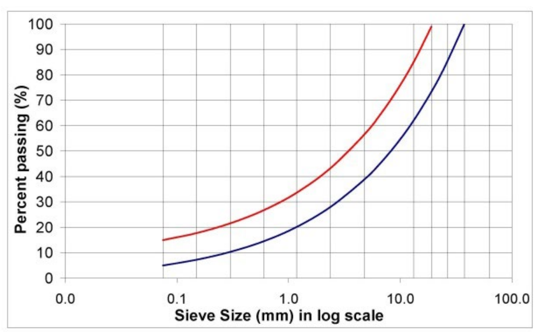



Figure 2: Ideal grading envelope for Foamed Bitumen treated material using the log scale for sieve sizes

### 3.4 Aggregate Pre-treatment

Section 9.3 of these notes gives guidance for the design engineer to the aggregate pre-treatment, if required. Pre-treatment is usually with lime or KOBM and is to reduce plasticity as an aid to the subsequent stabilisation treatment. Where pre-treatment is employed the mix design process shall be structured to match the construction process. Unless otherwise specified by the design engineer this will involve pre-treatment of the mix design sample which will then be bagged and left to condition for 24 ± 2 hours prior to undertaking mix design testing.

### 3.5 Optimum Water Content and Maximum Dry Density

The density, and therefore the strength, of compacted stabilised material is very dependent on the degree of compaction. Normal stabilising practice in the field is to assess the moisture content of the stabilised material by means of the squeeze test and the moisture content is adjusted as the stabilising operation advances. The squeeze test is a good practical method and usually gives the correct compaction moisture content on site, which is in the region of 90% to 100% of the Optimum Water Content (OWC). It is therefore important to simulate this in the laboratory. However for purposes of standardisation and for comparing various laboratory results the compaction moisture content in the laboratory needs to be standardised. Experience has shown that compaction of stabilised materials at between 85% and 95% of the stabilised material's OWC gives similar densities as those found in the field.

3.5.1 Squeeze Test method of determining the optimum compaction moisture content

(a) Pick up a handful of material
(b) Remove large particles (i.e. stones greater than 25mm)
(c) Squeeze the sample in the hand firmly
(d) Observe material:
  (i) If the sample is dull and falls apart: sample is too dry – add some water;
  (ii) If the sample is shiny and falls apart: sample too wet – dry by placing the sample on a tray and use a fan to dry. Retest every two minutes until the sample is close to its optimum compaction moisture content;
  (iii) If the sample is slightly dull but sticks together: the sample is close to optimum compaction moisture content.

Optimisation of the binder content for stabilised materials involves manufacturing test samples for indirect tensile strength (ITS) testing. The ITS test result is sensitive to the density and therefore standardisation of determining the compaction method is important.

When producing test samples for basecourse stabilisation the vibrating hammer compaction method (NZS 4402:1986 Test 4.1.3) is the recommended compactive effort. This method achieves densities comparable to those achieved in the field.

Note that the testing agency may elect to undertake the compaction of the ITS test samples using gyratory compaction. Provided that the gyratory compaction settings can be clearly demonstrated to produce test specimens of similar density (i.e. ±5% to that achieved by vibratory compaction for the material) this is considered to be acceptable.

In the case of subbase stabilisation, be it modified or bound, the design engineer needs to decide on the compaction method using the following guidance:

(a) If the anvil for compaction below the stabilised subbase is sound then the laboratory test samples should be compacted using the vibrating hammer compaction method (NZS 4402:1986 Test 4.1.3).

(b) If the anvil for compaction below the stabilised subbase is not robust then the laboratory test samples should be compacted using the standard compaction method (NZS 4402:1986 Test 4.1.1).

The equipment for these compaction tests is available in most aggregate laboratories in New Zealand. Where the vibrating hammer is not available or practical, e.g. for producing test samples in the field, the heavy compaction method (NZS 4402:1986 test 4.1.2) may be used.

## 4 TEST PROCEDURE FOR CEMENT AND/OR LIME STABILISED MATERIALS

The design engineer may wish to prepare and undertake an Unconfined Compressive Strength (UCS) test to further classify treated material properties. This is outside the scope of this specification.

Laboratory compaction should replicate achievable field compaction. As such, T19 refers to appropriate target densities for subbase and basecourse layers derived from NZTA B2. The design engineer can modify these but variation of these target densities should only be done after careful consideration.

## 5 TEST PROCEDURE FOR BITUMINOUS STABILISED MATERIALS

The design engineer may wish to prepare and undertake a UCS, Indirect Tensile Modulus or Flexural Beam testing to enable the inferred resilient modulus to be determined. This is outside the scope of this specification.

Laboratory compaction should replicate achievable field compaction. As such, T19 refers to appropriate target densities for subbase and basecourse layers derived from NZTA B2. The design engineer can modify these but variation of these target densities should only be done after careful consideration.

## 6 DETERMINATION OF THE INDIRECT TENSILE STRENGTH

Table 1 and Table 2 below give an indication of the ranges of dry Indirect Tensile Strength (ITS), soaked ITS and the Tensile Strength Ratio (TSR) that should be aimed for. The ITS range in Table 1 corresponds to compaction to 100% Maximum Dry Density (MDD) in accordance with NZS 4402, Test 4.1.3, New Zealand Vibrating Hammer. As more data becomes available to industry through research and construction, these values may be adjusted if required.

The reason for standardising the compactive effort is to provide consistency in mix design process rather than vary the target density based on assumed field compaction capability, which may introduce an error. Additionally, existing pavement materials may be variable and following this, the stabilisation (hoeing) process may change the grading – particularly if dual stabiliser passes are required as with pretreatment meaning that selection of an appropriate compaction target is extremely difficult at mix design stage.

It is recommended on this basis that the compaction protocol be standardised to NZS 4402, Test 4.1.3 New Zealand Vibrating Hammer. However, for major projects, particularly those utilising new basecourse, the design engineer may direct additional compaction targets (i.e. 95% or 98% Maximum Dry Density) in order to assess the impact on ITS properties of reduced density.

Table 1: Guidance for Indirect Tensile Strength (ITS)

| Pavement Layer Type | Dry ITS (kPa) | Soaked ITS (kPa) |
|---|---|---|
| Cement / Lime Modified Basecourse | 150 to 350 | 100 to 300 |
| Foamed Bitumen Stabilised Basecourse | 175 to 400 | 150 to 350 |
| Cement Bound Subbase | > 500 | > 450 |

Note that the values given in Table 1 are based on the ITS being determined using the 1mm/min strain rate. Extensive testing has previously been done with faster strain rates (in particular 50.8mm/min) and calibration testing indicates that there is a "shift" in the results. The shift is in the order of 20 to 40% higher ITS for Foamed Bitumen Stabilised (FBS) test specimen at 50.8mm/minute. If other strain rates are used then these will need to be adjusted by means of some calibration testing.

At the time of writing T19 several laboratories have traditionally undertaken foamed bitumen mix design specimen ITS testing at the 50.8mm/min loading rate which is specified by the South Africa Asphalt Academy (in TG2 Technical Guidelines: Bitumen Stabilised Materials, second edition 2009), and the Wirtgen Cold Recycling Technology (1st Edition 2012). On this basis, it is anticipated that the only typical departure from 1mm/min loading rate would be to utilise 50.8mm/min for which there is a large body of mix design and construction quality assurance testing data available since 2006.

Table 2: Guidance for Tensile Strength Retained (TSR)

| Terrain type and drainage | Tensile Strength Retained (TSR) | | |
|---|---|---|---|
| | Dry (Rainfall < 600 mm/annum) | Moderate (Rainfall between 600 and 1000 mm/annum) | Wet (Rainfall > 1000 mm/annum) |
| Rolling / well drained | > 50% | > 60% | > 70% |
| Flat / poorly drained | > 60% | > 65% | > 75% |

Note: field validation of in situ bituminous stabilised materials is sometimes requested in the form of coring. Foamed bitumen stabilised materials are non-continuously bound and are sensitive to disturbance during curing and without oven curing take some time to develop tensile capability through field curing and dry-back. The ability to core bitumen stabilised materials without disturbance increases significantly after a period of 6 to 8 weeks.

## 7 OPTIMISING THE FOAMED BITUMEN CHARACTERISTICS IN THE LABORATORY

The objective is to determine the percentage of water that is required to optimise the bitumen foaming characteristics, being expansion and half-life. The aim is to produce foamed bitumen with the largest possible expansion ratio while maintaining the longest possible half-life.

Expansion is defined as the maximum increase in volume relative to the original bitumen volume. Half-life is defined as the time, measured in seconds that the foamed bitumen takes to collapse from the maximum expansion to half of the maximum expansion. These terms are represented in the schematic below.

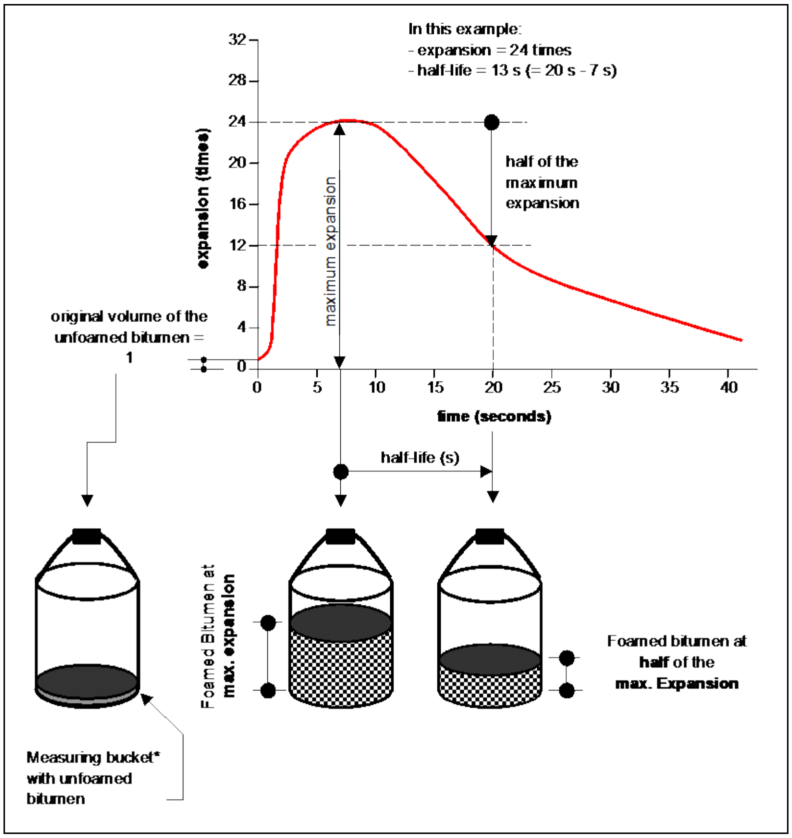



Figure 3: Explanation of expansion and half-life

The curve above represents a typical progression of expansion and half-life of foamed bitumen.  Each point of a foamed bitumen optimisation graph (Figure 3) is done with 500 g of bitumen into a 20-litre steel-measuring bucket.

## 8 USEFUL REFERENCES

(a) Austroads Guide to Pavement Technology, Part 5: Pavement Evaluation and Treatment Design
(b) Austroads Guide to Pavement Technology, Part 4D Stabilised Materials
(c) Wirtgen Cold Recycling Technology (1st Edition 2012)
(d) TG2 Technical Guidelines: Bitumen Stabilised Materials, second edition 2009
(e) NZ Geotechnical Society “Field Description of Soil and Rock (3)
(f) NZTA B5: Specification and Notes for In-situ Stabilisation of modified pavement layers
(g) NZTA B6: Specification and Notes for In-situ Stabilisation of bound subbase pavement layers
(h) NZTA B7: Specification and Notes for the manufacture and construction of plant mixed modified pavement layers
(i) NZTA B8: Specification and Notes for the manufacture and construction of plant mixed bound sub-base pavement layers

# 9 APPENDIX A: CHARACTERISATION OF SAMPLED MATERIALS AND BINDER SELECTION – A GUIDE FOR THE DESIGN ENGINEER

## 9.1 Evaluation of the Test Pit Investigation

From the Test Pit log and Scala penetration profiles, the design engineer will generally be able to identify the most likely rehabilitation option and estimate the overlay thickness, if required and the stabilisation thickness. The design engineer may then ask for further testing of the sampled aggregate in order to confirm these first assumptions before specific indications are provided on which mix design has to be done.

## 9.2 Aggregate characterisation testing

As a minimum, the design engineer should commission the following tests to be carried out on pavement and imported materials:

(a) wet sieve analysis of the aggregate in accordance to NZS 4407 Test 3.8.1
(b) plasticity index (PI) of the aggregate in accordance to NZS 4407 Test 3.4
(c) soaked CBR in accordance to NZS 4407 Test 3.15 on the aggregate from each individual pavement layer below the rehabilitation depth and on the subgrade material using the following surcharge weights:
  (i) Basecourse: no surcharge (unless deep lift asphalt which may require loading);
  (ii) Subbase: weight (kg) = thickness of basecourse (mm)/130mm rounded to the nearest kg;
  (iii) Subgrade: weight (kg) = thickness of pavement layers (mm)/30mm rounded to the nearest kg.

The results are primarily used for material classification, which provides an indication of relevant parameters (e.g. elastic modulus) for use in analysing the existing pavement structure and designing the rehabilitated pavement structure, starting from the assessment of their compatibility with different stabilising agents.

Not all of the above tests need to be performed, and generally not on each test pit sample separately. However it is strongly recommended that sufficient wet sieve grading and Plasticity Index (PI) testing is undertaken to categorise the range of aggregate properties.

At this point the design engineer should discuss the test regime with the laboratory technician based on the consideration that the basecourse and sub-base is normally sufficiently homogeneous to allow for the grading, PI and California Bearing Ratio (CBR) to be determined on an averaged sample from all (or selected) test pits. If they are not, a “single point” mix design approach may be preferable.

Particle Size Distribution testing (grading) will normally be done on a washed and oven dried sample to NZS 4407 Test 3.8.1. If, however there is a presence of bitumen bound aggregate (multiple seal or thick asphalt, up to 100% RAP), the sample must be dried at lower temperature to avoid the bitumen softening. Note that for the mix design, the grading is always done on the cold material without extracting the bitumen.

CBR of the sub-base aggregate can be waived for well graded materials or when the coarse fraction is predominant (high internal friction). However, the compaction of a remoulded sample may be required to estimate the in-situ density (unless it can be differently estimated), so that a CBR can be performed on it. Unless the test pit log shows that layers where in a "loose" condition (which is generally quite unusual), each material will be compacted as close as possible at the Optimum Water Content (OWC), not at its natural moisture content.

In case of an overlay material being incorporated, the imported material needs to be sampled and tested.

Should the aggregate look similar to others that are known by previous experience to be reactive, or not reactive, to specific binders/contents under specific conditions, this shall be reported to the design engineer. The design engineer can use historical tests results for initial evaluation purposes, but the material should always be tested for a confirmation before it is used in the mix design.

### 9.3 Pavement design and binder selection

From the test pit information the design engineer can start assessing the various feasible stabilisation options. The Austroads Guide to Pavement Technology Part 4D: Stabilised Materials (1) gives good guidance as to what binder can be used for the particular aggregates found.

In New Zealand the most commonly available binders as follows:

- (a) Lime to NZTA M15 specification, hydrated lime ($Ca(OH)_2$) or quicklime ($CaO$). Note that for health and safety reasons it is common to use hydrated lime in the laboratory. If, quicklime is used either in the laboratory or in the field then it needs to be thoroughly slaked before adding to the aggregate;
- (b) Cement, NZS 3122 – normal Portland (GP) cement;
- (c) Foamed Bitumen: NZTA M1 or M1-A (as base bitumen for foaming).

At this stage the design engineer needs to decide on:

- (a) the depth of stabilisation, which will determine the blend of materials to be used;
- (b) the type and range of binder contents to be used in the mix design (see Table 3 below);
- (c) the thickness, type and source of the aggregate (AP/GAP/sand, 20/40mm, quarry/borrow pit) or imported overlay material, if required.

The design engineer will have to specify at least:

- (a) type of binder;
- (b) dosages of each binder;
- (c) how the binders are to be blended into the aggregate (in case of pre-treatment);
- (d) any time delays between mixing and compaction (in case of in-plant mixing).

Note: where more than the insitu basecourse aggregate is to be tested for mix design (e.g. basecourse, subbase, overlay and/or surfacing), define the source aggregate, additional aggregate and surfacing type and proportions to be blended and use this blend for the mix design (refer section 9.4)

The binder content and type tested in a mix design are quite variable as they depend on many factors. However, usually three contents around the likely optimum are normally tested. Typical binder contents tested for basecourse stabilisation are given in Table 3below.

Table 3: Suggested Binder contents for mix design purposes:

| **Stabilised layer type** | **Material to be treated** | **Binder** | **Suggested Binder content range** |
|---|---|---|---|
| Cement modified or lightly bound | Aggregate with PI < 15% | Cement | 1, 1.5, 2% |
| Lime modified or lightly bound | Marginal aggregate with 15% < PI < 20% | Lime | 2, 3, 4% |
| | Very Marginal aggregates, PI > 25% | Lime | 3, 4, 5% |
| Heavily cement bound | Aggregate with PI < 10% | Cement | 4, 6, 8% |
| Foamed Bitumen modified | Aggregate with PI < 15% | (A) Foamed Bitumen and Cement | 2.7 FB + 0.8 Cement<br>2.7 FB + 1.0 Cement<br>3.0 FB + 0.8 Cement<br>3.0 FB + 1.0 Cement |
| | | (B) Foamed Bitumen and Lime | 2.7 FB + 1.0 Lime<br>2.7 FB + 1.2 Lime<br>3.0 FB + 1.0 Lime<br>3.0 FB + 1.2 Lime |
| | Aggregate with PI > 15% | Lime or KOBM pre-treatment | 2 - 3% |
| | | Foamed Bitumen with Cement or Lime | As above for (A) and (B) |

**Notes:**

(a) the values in Table 3 are given as a guide only and should always be verified by the pavement design engineer;
(b) the binder dosages in the laboratory are calculated as percent of aggregate passing the 37.5 mm fraction while in the field they are calculated as percent of the whole material;
(c) Table 3 refers to quicklime
(d) when using lime, always specify if it is hydrated lime or quicklime;
(e) when using quicklime, remember that:
  (i) it needs to be slaked if modification of the plasticity is required;
  (ii) it has a much stronger drying effect (real or apparent) than other binders;
  (iii) a "reaction" period between mixing and compaction is required;
  (iv) quicklime has about 30% more effective calcium than hydrated lime;
(f) the choice of binder type and content may sometimes depend on reasons other than aggregate/binder compatibility; for example, the design engineer may decide for a cement only mix design on a material whose high PI and fines content would otherwise suggest a lime pre-treatment, purely for reasons of availability and/or economics.

It is recommended that the design engineer explains the choice of binder so that the technician can verify that it is not due to a misinterpretation of the laboratory test results.

Report blend combinations (i.e. cement and lime, cement and KOBM etc )

Increasing the cement content of bituminous stabilised mixes increases the likelihood and risk of cracking.

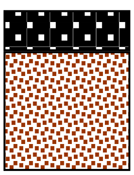
### 9.4 Blending of Aggregates

Produce sufficient quantity of aggregate to carry out all the tests required by the selected mix design by blending the aggregates sampled from the single pavement layers, plus any imported aggregate if required.

The design engineer specifies the volumetric proportions (layer thicknesses) and the laboratory technician converts them into dry mass proportions so that they can be used by the laboratory. The in-situ density of the various materials must be considered when blending them to reproduce a sample representing the full recycling depth. It can be estimated from experience or from the dry density and the in-situ moisture content from the test pit log. In New Zealand the majority of roads (particularly rural) involve pavement layers where the basecourse and sub base aggregates consist of similar aggregates with chipseal surfacing. In these cases one can assume that these are the same density (the error is minimal if compared to other approximations).

However, in some instances the stabilisation involves a significant proportion of asphalt or multiple seal layers. In these cases the in-situ density should be taken into account as shown in the following example:

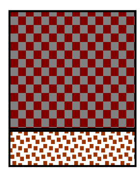
**Existing upper pavement structure**



60 mm Asphalt
(in-situ density 2300 kg /m$^3$)

200 mm AP 40 Basecourse
(in-situ density 2100 kg/m$^3$)

**Stabilised upper pavement**



200 mm Stabilised Basecourse
(60 mm Asphalt + 140 mm AP 40)

The materials are blended in proportion to layer thickness and in-situ density as follows:

| Material | Per square meter (kg) | Proportion by mass (%) | Per 10 kg sample (g) |
|---|---|---|---|
| Asphalt (60 mm at 2300 kg/m$^3$) | 0.06 x 2300 = 138 | 138 / 432 = 0.32 | 0.32 x 10,000 = 3,200 |
| AP 40 (140 mm at 2100 kg/m$^3$) | 0.14 x 2100 = 294 | 294 / 432 = 0.68 | 0.68 x 10,000 = 6,800 |
| Total | 432 | 1.00 | 10,000 |

### 9.5 Aggregate / binder mixing

The laboratory mix design should replicate as closely as possible the process that takes place in the field. Therefore all stabilised mixes should be produced using a suitable mechanical mixer. This is especially true for foamed bitumen mix designs as it is the only practical and safe way of adding the foamed bitumen to the aggregates. It should also be noted that research has found that the mixing energy of a laboratory scale pug mill mixer simulates the mixing capability of modern stabilisers more accurately. The design engineer should make the laboratory aware of any project requirements that may influence the mixing process.