I FACOLTÀ DI INGEGNERIA

Master of Science in Civil engineering

Master's thesis

Evaluation of the use of a site-won gravel as a Hydraulically Bound material in a Base layer of a pavement

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December 2017

To my family,
my guide.
To myself,
for always having believing in it.

ABSTRACT

In the UK, sustainability and development are the key words for the construction system of the pavement.
The aims of the use of Hydraulically Bound Mixtures (HBM) are to minimize the emission during the constructive steps, to reduce the time of the installation and to improve the operating costs, by guaranteeing a great resistance during the time.

HBM can be defined as mixtures which comprise aggregates with controlled grading and an hydraulic binder. When the hydraulic binder is the cement, it is called Cement Bound Granular Material (CBGM).
This material can be used as pavement base layer. This layer is designed to distribute the stresses and strains exerted from the passing traffic loads down to the foundation.

The BS EN 14227-1:2013 provides specifies CBGM for roads, airfields and other trafficked areas and specifies the requirements for their constituents, composition and laboratory performance classification. The main specification under consideration is Series 800 of the Manual of Contract Documents for Highway Works [MCHW1, 2005].

The aim of this investigation was to evaluate the use of the site-won gravel as a HBM in a base pavement layer and analysing the pavement design under these conditions.

According to the Series 800, the aggregates used in CBGM were tested in order to comply with the requirements. The proportions of the constituents, including water, were determined based on the mixture design procedure described in the Clause 880 [MCHW1, 2005].

To determine the mechanical performance specified in the Clause 870 [MCHW1, 2005] the properties of the CBGM were determined at a minimum of 3 values of binder contents, and a minimum of 2 values of water content for each of them.

The best mixture was selected from the results obtained by the laboratory investigation. The mix design results showed a low strength material which did not comply with the minimum requirement strength for a base layer.

Based on this final result, it was decided to analyse the pavement design using this material to verify and to understand the reason why it is not suitable for the use in a base layer.

The currently predominant procedure in use for pavement design in the United Kingdom is the Design Manual for Road and Bridge 2006 Part 3 HD 26/06 guide. In fact, the required thickness for a specific foundation class, material and traffic can be found out through the nomograph provided by this guide.
Accordingly, Bisar 3.0 was used to determine stresses within the pavement for developing a design methodology.
The first step was to ascertain Bisar adequacy and establishing inputs related to loading, thickness and subgrade condition. Using the equations suggested by HD 26/06, an iterative process was used to calculate the base layer thickness to ensure a specific level of traffic.
By following this process, the curve relating to CBGM was plotted on the nomoghraph.

With this analysis, it was highlighted how different thickness of CBGM with a low strength shows up compared to CBGM with the minimum class of resistance. In conclusion, an high thickness must be used with this material confirming that these aggregates are not suitable for being used as CBGM in the pavement base.

ACKNOWLEDGMENTS

I would like to thank the company AECOM, in Nottingham, for giving me this work experience opportunity.

In particular thanks to J.Tuck to provide me this topic. Thanks for her support and assistance.

My gratitude even goes to P.Edward for his invaluable contribution to the development of this project. I will treasure his help, suggestions and teaching.

The same appreciation goes to H.Lacalle, D.Casey and P.Lopez for their encouragement, help and continuous support.

Even thanks to M.Jones for his technical support. His help was essential for carrying out my research.

I also express immense gratitude to all colleagues and to all technicians who assisted me with the laboratory work. Thanks to their sincere help and to support me in each step of this work.

I owe thanks to my supervisors E.Santagata to provide this internship, without which this research and my experience in the United Kingdom would be difficult to achieve.

Finally thanks to all people who I met during this experience. Thanks for their friendship that I hope it will last across time.

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INTRODUCTION

This Thesis presents work carried out at AECOM Nottingham during a secondment of 7 months, between May and November 2017. The aims of this secondment were to gain knowledge and industrial experience working directly in AECOM's commercial projects (May to July) and to work on a research project which could be presented as a master's thesis.
After the first 3 months working on several projects, it was decided to focus the research on the development of a pavement design using innovative materials.
In July 2017, AECOM Infrastructure & Environmental UK Limited were commissioned by a client to undertake a mixture design for a Specification for Highway work, Series 800, Clause 822 Cement Bound Granular Material (CBGM B) using site-won gravel, in a base pavement layer, stockpiled near London.
The aim of this investigation is to evaluate the use of the site-won gravel as a HBM in a BASE pavement layer and analyse with these conditions the pavement design.

SCOPE AND BACKGROUND TO THE PROBLEM

Standard pavement design does not allow the use of materials in pavement layers when certain characteristics and mechanical properties are not met. This is a limitation when materials are available on site but do not meet the requirements set in the specification.
To avoid disposing material with potential to be use in pavement layers, these materials can be characterised and mixed in a laboratory, a mix design appropriate for pavement layers can be established and a pavement design thickness can be calculated.

In this research, the possibility to use site-won gravel as base course is investigated. To be able to use this gravel as a base course it was decided to study the possibility of obtaining a Hydraulically bound material (HBM) using the gravel as aggregate and cement as hydraulic binder.

The most commonly used aggregate in these types of mixtures is crushed rock, as it gives better mechanical properties. In this regard, Macadam and Telford, in the early 1800's, were the first engineers that recognise the importance of mechanical behaviour of pavement materials to provide good pavement performance. They agreed that using angular aggregates or aggregates with crushed faces improve the pavement performance as the aggregates interlock better (Dawson, 1995). This

shows a clear link between the understanding of material behaviour and successful pavement construction (Edward , 2007).
However, the use of this type of aggregates is not always possible. There are some circumstances where using other type of aggregates is more economic and viable; therefore, the pavement design should be modified to include this type of aggregate.

Nevertheless, there are some risks associated with the use of other aggregates different from crushed rock. The primary risks associated with the use of rounded gravel in HBM mixes are as follows:

- The use of gravel gives a more brittle mixture compared with the one obtained with crushed rock, leading to an increased risk of cracking.
- It is also possible that the flexural strength is reduced compared with crushed rock HBM.
- If a crack develops, having rounded rather than angular aggregates reduces the aggregate interlock, facilitating the crack propagation. .
- Gravel may also have high fines content and may require increased binder content or additional processing.
- In terms of compaction, surface regularity can be more difficult to achieve with rounded gravel compared with angular crushed rock.

For these reasons, it is very important to analyse the mechanical and physical characteristics of the material in order to ensure the durability of the pavement layers.

A pavement is probably one of most complex structure designed and constructed by engineers. This is because the layers which form the structure are of different materials and different thicknesses (Stock, Phil, 1979).
It is interesting to understand how the characteristics of the materials used and the traffic information are related to the layer thickness.

AIM AND OBJECTIVES OF THE PROJECT

The aim of this research project is to analyse the suitability of a site-won gravel to be used as aggregates of a HBM mixture for a base course. With this purpose, three main objectives were established, related to better understand the properties of this type of material, in order to be able to predict its performance, and to establish a design procedure. The three objectives can be summarised as:

- Classifying the material, its geometry and mechanical properties

- Develop a testing procedure to manufacture and characterise the mechanical behaviour of the HBM mixture;

- Develop a design methodology which predicts the layer thickness needed with this material to withstand different loads during the life of the pavement.

1. LITERATURE REVIEW

When we talk about roads, the thought is to the ancient romans and their engineering: they threw the bases of the recent construction techniques of the roads scientifically moving in the choice of the layouts to follow and in the materials for the superstructure's realization (Packer J.I, 1980). (D.X. Xuan, 2011)
The construction of the roman roads began with the excavation of two deep and parallel trenches ("sulci") that defined the width of the road.
The incoherent soil between two sulci was removed digging until a level of stable ground was reached on which the pavement could be constructed (Mascio, 2010)

The roman pavement consisted of four different layers (J.C, 1997):

- The "*statumen*", now called sub-grade, formed of big stones that could be placed by hand;
- The "*ruderatio*", now called base, consisted on stones and broken aggregates with an activator, i.e lime;
- The "nucleus", now made up of surface and binder course,made of aggregates smaller than the previous layer. It was the layer on which the pavimentum was based.
- The "*pavimentum*", or use-surface, prepared with "basoli", big stones with flat surface, similar to the actual paving blocks, and V shaped in order to facilitate drainage.

Figure 1 shows a typical cross section of a roman pavement.

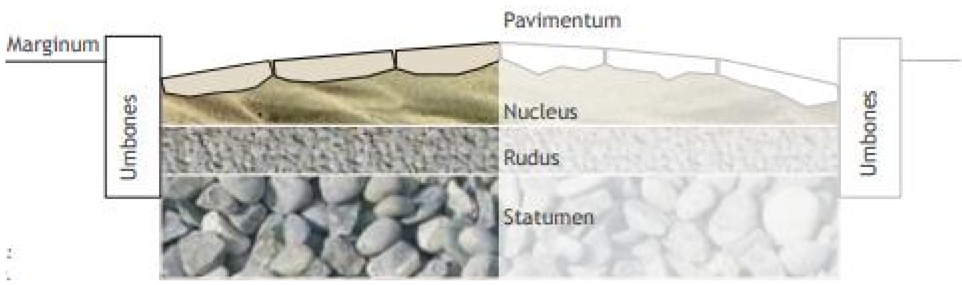


Figure 1 Roman pavement (Mascio, 2010)

Nowadays different materials are used to build a pavement. The chemical, physical and mechanical characteristics are different but the roman heritage is still evident.

To understand how a pavement works, how it is design and how it performs, it is important to understand its functions, structure and materials used.

1.1 DEFINITIONS AND PAVEMENT FUNCTIONS

A pavement is defined in civil engineering as the durable surfacing of a road, airstrip, or similar area, and its primary function is to transmit loads to the sub-base and underlying soil. A typical pavement structure is based on layers and these layers can consist of different materials according to their purpose (H., 2017).

Generally, the primary functions of a pavement are (SANRAL):

- assure a reasonably smooth riding surface*:* it is essential to give riding comfort. During the years, the users measure the quality of the road through this parameter. Possible causes as the structural deformation can leads phenomenon of roughness. This is a parameter used to valuate type, cost of maintenance end timing needs.
- provide sufficient skid resistance*:* it is important both for the road users and for a riding comfort. The surface friction between the tyre and the surface of the pavement is an important point to assure the user safety. The pavement condition can affect more this problem especially during wet conditions. In general, the skid resistance must be provided in all pavement conditions in order to ensure the safety.
- protect the subgrade*:* The subgrade is considered the pavement layer which has a supporting pavement function. The applied load can over-stress the subgrade causing its deformation and loss of ability to correctly support these loads. Therefore, the pavement must have sufficient structural capacity in terms of strength and thickness to sufficiently reduce the stresses on top of the subgrade. Depending on the type of subgrade and on loading conditions (magnitude and number of axle loads) the strength and thickness requirements of a pavement can vary.
- offer waterproofing*:* If the water gets into the underlying layers can transform its saturated. This means that the soil and the other layers lose the ability to support the applied load and in some case, can happen the premature pavement failure

1.2 PAVEMENT STRUCTURE

Figure 2 illustrates a typical pavement and pavement foundation structure. On top, the **surface course** is a high-quality and relatively expensive material. It is designed to withstand direct loading and to provide adequate skid resistance. The most commonly used surface course material in the UK is asphalt. The **binder course** can be similar to the base; however, it has a smaller aggregate size. The **base** gives the pavement most of its strength and it can be asphalt, hydraulically-bound material or granular (Thom, 2008).

The **sub-base** layer's function is to limit pavement flexure and the performance of an asphalt or concrete base depends critically on the stiffness and quality of this layer (R.N, 1994).
It normally comprises hydraulically-bound or granular material. The **capping** is generally a lower cost materials and its purpose is to have a suitable layer to allow safe construction over the subgrade. In this layer, either hydraulically-bound or granular materials are commonly used (BACMI, 1992).
Finally, the **subgrade** is the natural (or imported) soil that has to be protected from applied loads and contamination by the previous layers (Thom, 2008).

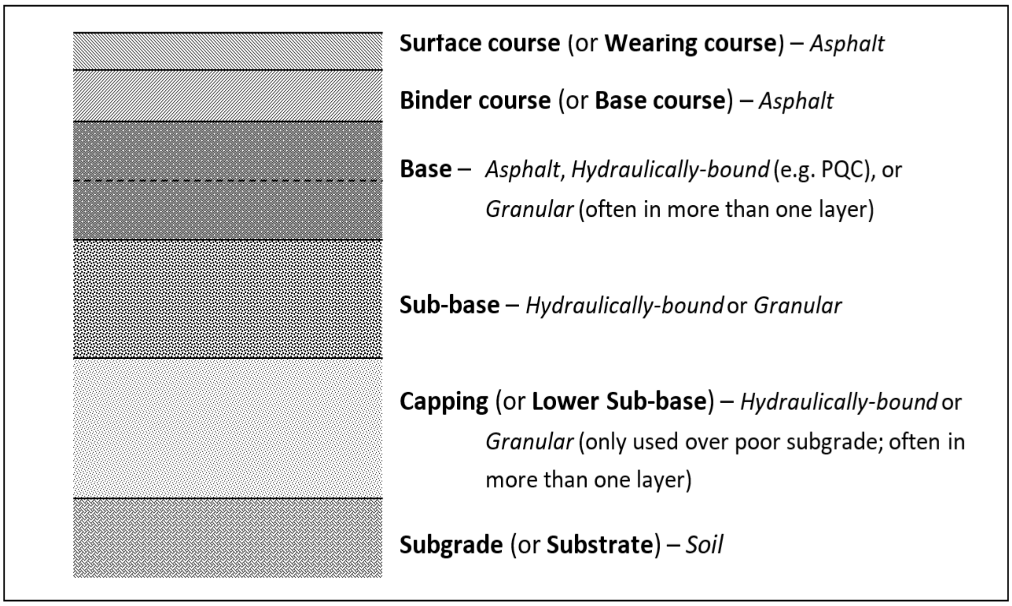


Figure 2 Pavement layer (Thom, 2008)

1.3 PAVING MATERIALS

The material for pavement construction can be classified as:

- Soils: they are within every pavement. Protection from applied loads is one of the most important requirements for underlying soil. Soils are sensitive to water content, highly permeable and can contain clays (deemed to be enemies for the soils). Using additive such cement and lime it is possible to improve its properties.

- Granular material: includes natural gravel, crushed rock and granulated industrial by product. Granular materials are usually large particle size and it can also be mixed with water to improve the characteristics. In confront of soil this material is much more controlled and predictable component.
- Hydraulically Bound Material: is a mixture of aggregates and binding agent. It requires water in order to take place the cementing action. The stiffness can depend on the contact between large particles and on the quality of the cementious matrix. Next part will discuss all properties an associated test.
- Bitumen-bound material: defined as tarlike mixture of hydrocarbons produced by the distillation of crude oil petroleum refining (IARC,1985). Know like asphalt, currently play a major role in the centre of pavement technology. Bitumen is a binder characterised by a viscous liquid aspect in service temperatures. It can flow and this tendency may seem undesirable because can lead phenomenon known as “rutting” or “tracking” in which the asphalt can deform (Thom, 2008).

Other materials are used to improve the strength of the pavement structure as block paving, hybrid material, steel reinforcement and geosynthetics.

As this Thesis focuses on the development of a hydraulically-bound material as Cement Bound Granular Material type B. This is more extensively described.

1.4 HYDRAULICALLY BOUND MIXTURE

A Hydraulically Bound Mixture (HBM) can be defined as a mixture comprising aggregates with a controlled grading and a hydraulic binder(s) that has been mixed using a technique that produces a homogenous mixture [adapted from BS EN 14227-1, 2004]. For hydraulic binder means that hardens by a chemical/hydraulic reaction with water.

Depending on the type of hydraulic binder used in the mixture, HBMs are classified in to three groups (WRAP, 2005):

- Cement Bound Granular Material **(CBGM)** (BSEN 14227-1). CBGM require a binder such as cement and water. It is used in road, airfield, port, and other heavy-duty pavement construction projects. It is an affordable and a great sustainable alternative to asphalt construction decreasing the cost of the project. The cement bound granular material is used worldwide to improve the properties of base’s road and airport pavements
- Fly Ash Bound Material **(FABM)** (BSEN 14227-3). Fly Ash bound mixtures are used in the construction of sturdy pavements. It is also used in the road construction because it is versatile and high quality performing.

- Slag Bound Material **(SBM)** (BSEN 14227-2). It uses the hydraulic combination of granulated blast furnace slag and air-cooled steel slag/concrete. It is widely used in the construction of highways where thickness and strength is required.

Depending on the type of the binder used the reaction speed change. There are quick or slow setting binders. The first are characterized by fast setting and hardening like cement. The second have slow setting as fly ash or furnace slag.

When Portland cement is mixed with water and aggregates, the hydration reaction improves the mechanical characteristics of the material over time.
It is needed a curing time in which the mixture is protected from loss of moisture increasing strength and decreasing permeability.
Research studies have shown that the mixture cement-water-aggregates increases in strength very quickly for a period of 3-7 days until to reach about 90% the strength required into 28 days as shown Figure 3 (Quora, 2017).

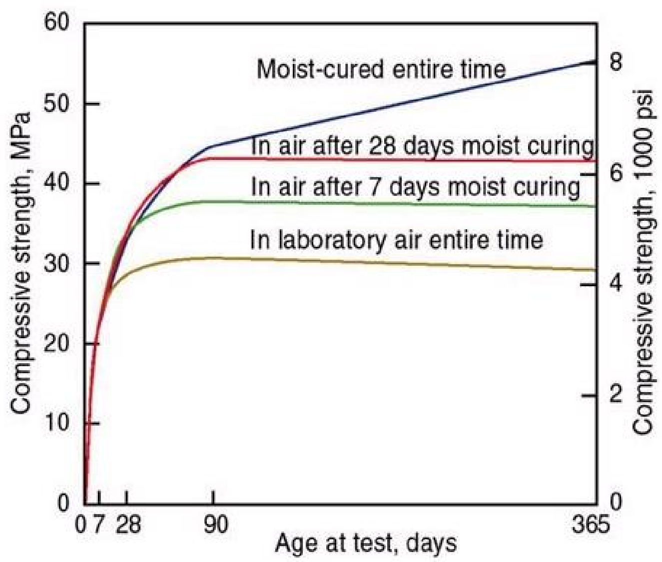


Figure 3 Tipical streigth-gain curve (Quora, 2017)

For this reason, 28 curing days are required.
During this time, all hydration reactions shall be completed.
The rate of strength gain, the ultimate strength and the overall performance of an HBM depends on its age, curing time and conditions and its composition (percentages of binder and aggregates) (D.X. Xuan, 2011).

The aggregates used for the mixture can be grouped into three broad categories defined by their source (WRAP, 2005):

- primary aggregates - aggregates produced from naturally occurring mineral deposits that are being used for the first time;
- secondary aggregates - usually by-products of industrial processes that are being used for the first time; and
- recycled aggregates - derived from reprocessing materials previously used in construction, such as demolition material (crushed brick, concrete and ceramics) and railway ballast. This can include demolition material containing primary, secondary or recycled aggregates. For example materials from construction and demolition waste are used like up-cycle material.

These materials mixed with cement provide the basis for Hydraulically Bound Mixture.

Hydraulically Bound Materials (HBMs) are one of the most commonly used materials for pavement sub-bases and bases (Britpave, 2005).
The sub-base layer, which is very important in terms of the expected performance of the pavement. The sub-base is often the main load carrying layer of the pavement, as opposed to the base, and is designed to distribute the stresses and strains exerted from passing traffic loads down to the foundation.
The base, which needs to be strong to prevent shear and structural failure, as opposed to surface deformation, also known as rutting, in the overlying asphalt layer. In addition to providing strength, a well designed and constructed base will provide good drainage and prevent settlement (WRAP, 2005).

In pavement design, the sub-base is usually unbound whilst the base is the lowest of the bound layers. However, the two types of materials which are most commonly used in the sub-base and base layers are unbound granular materials and hydraulically bound materials. Aggregates used in either type of material for these layers usually contain good interlocking properties; thereby, traffic loads are evenly distributed through the layer and the underlying layers. Crushed aggregates are generally recommended for these materials (WRAP, 2005).

HBMs can be used in a wide range of applications including working platforms, liners, erosion protection, major roads, minor roads, paved areas and heavy duty paving.

The production can take place in-situ or ex-situ. The in-situ method processes recycled aggregates on site. Binder is spread on soil and mixed with machinery.
HBM must be compacted quickly as a single layer in order to have fast setting.
The ex-situ method is performed in a central plant and requires treatment of the material at another location (Nicholls, 2016).

Recycled materials and aggregates are taken from stockpiles and mixed with binder (such as lime, cement or fly ash) and water and then transported to site for laying and compaction.

The principal advantages of using HBMs are the following (WRAP, 2005):

- HBM construction is popular and versatile in terms of availability of plant and materials.
- HBM can be produced "ex-situ" or "in-situ".
- The use of HBM is energy efficient as it is mixed at ambient temperature instead of the 170-180° needed for hot mix asphalt. In addition, by-products of local power stations and metal works, such as fly-ash and slag, can be incorporated in HBM design.
- HBM has become an attractive solution in several projects because of its green procurement which meets many clients' requirements.
- HBM strength and stiffness increases with time especially with HBM mixtures which are slow setting and slow hardening.
- Compaction process time is reduced when compared with unbound mixtures, which also significantly reduces health and safety issues related to hand arm vibration syndrome. The absence of the binder in the un-bound mixture requires more compaction process time in order to reach the same strength values.
- HBMs can be used to up-cycle waste material which is initially not suitable for pavement construction.
- The use of an HBM sub-base or base can significantly reduce the required thickness of pavement layers because of its high strength.

HBMs have even disadvantages like:

- Property requirements for the aggregate are selected by the user and are considered "open" in regards to strength, so that a poor choice as to the appropriate class of strength can lead to premature failure.
- Lack of binder in HBM design contributes to poor strength and susceptibility to frost heave.
- There is limited research on the use of HBM in trench reinstatements, particularly in weak pavements or roads in poor condition.

There is also another important problem connected to the use of HBM.
Generally the pavement will be less affected by fatigue and permanent deformations located on the surface layer.
It is important to point out that the hydration phenomenon may cause physical removal and subsequent cracking.

Fast setting HBMs provide good strength after initial construction but are prone to shrinkage during curing and, therefore, experience thermal stress cracks which, in turn, lead to reflective cracking in the asphalt surface.
In fact tensile strength has a very important role in reducing cracks during the hydration phase, but even during its life the layers are affected by bending forces due to traffic loads. The cracks shall be reflected on the upper layers and in several cases reaching the surface. The phenomenon of cracking, in addition to decreasing load bearing capacity and an increase in the deterioration process, promotes water damage in the pavement compromising the frost heave durability (Collis, 1993).

Several studies had been conducted about the possibility of reducing cracking in HBMs. Some of the possible solutions are:

1. Check the maximum quantity of clay. This substance may contribute adhesion problem between aggregates and binder;
2. Control the quality of the material (specially the maximum density, optimum moisture content and uniformity of the mixture);
3. Increase the thickness of the asphalt layers overlying HBM layer;
4. Reduce cement content.

1.5 CEMENT BOUND GRANULAR MATERIAL

Like just discussed CBGM are a type of HBM and in United Kingdom CBGM is used as material for pavement construction.
Depending on aggregate's selection and its characteristics, CBGM mixtures are designated as follows Table 1 Type of CBGM:

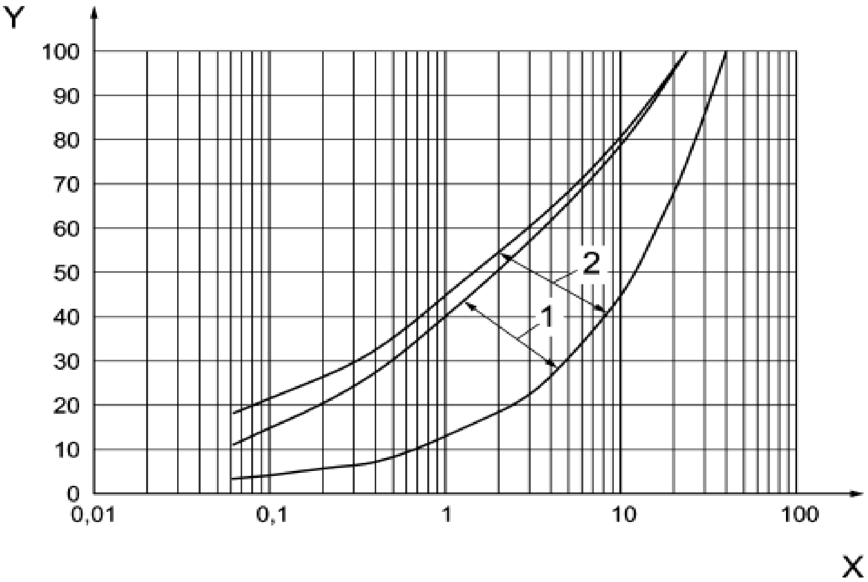
Table 1 Type of CBGM

Type of mixture	Suggested designation
CBGM with permitted grading envelope 'A' for aggregate. This covers wide-graded mixtures encompassing sand mixture made from either crusher run, as-raised materials or demolition aggregates etc.	CBGM A
CBGM with permitted grading envelope 'B' for the aggregate. This produces a 31.5 mm well-graded mixture	CBGM B

CBGM with tightly-graded mixture requirements producing either 20,14 or 10 mm well-graded mixtures	CBGM C 0/20 or C0/14 or C 0/10

In order to produce a high quality CBGM it is really important to verify if the stockpiled aggregates comply with the requirements of EN 14227-1.

For CBGM type B, in accordance with BS EN 14227-1, the percentage by mass of the mixture passing has to comply with the following grading envelope curve signed with the number 1 in Figure 4.



X sieve size, in mm

Y percentage of the mixture passing by mass

1 category G1

Figure 4 Grading envelope curves

Specifically, CBGM type B is a mixture that shall be either a 0/31.5 mm, a 0/20 mm or a 0/14 mm mixture with a grading.
The mixture with a 0/31.5 mm shall have a combined grading that complies with these specifics percentages of the mixture passing by mass shown in a Table 2:

Table 2 Grading of CBGM B

Sieve [mm]	Percentage of the mixture passing by mass	
	Minimum	Maximum Category G1
40	100	
31.5	85	100

25	75	100
20	65	94
10	44	78
4	26	61
2	18	50
0.5	8	30
0.25	6	22
0.063	3	11

The cement binder has an important role in this mixture.
Cement shall comply with EN 197-1 in which there is the followed definition:
” *Cement is a hydraulic binder, a finely ground inorganic material which, when mixed with water, forms a paste which sets and hardens by means of hydration reactions and processes and which, after hardening, retains its strength and stability even under water*”.
Different types of cement shall be used in order to produce the mixture:
-CEM I (Portland cement)
-CEM II (Portland compound cement)
-CEM III (Blast furnace cement)
-CEM IV (Pozzolan cement)
-CEM V (Composite cement)
The type of cement depends on the percentage in mass of the constituents as shown Figure 5.

Main types	Notation of the 27 products (types of common cement)		Composition (percentage by mass[a])										
			Main constituents										Minor additional constituents
			Clinker	Blast-furnace slag	Silica fume	Pozzolana		Fly ash		Burnt shale	Limestone		
						natural	natural calcined	siliceous	calca-reous				
			K	S	D[b]	P	Q	V	W	T	L	LL	
CEM I	Portland cement	CEM I	95-100	–	–	–	–	–	–	–	–	–	0-5
CEM II	Portland-slag cement	CEM II/A-S	80-94	6-20	–	–	–	–	–	–	–	–	0-5
		CEM II/B-S	65-79	21-35	–	–	–	–	–	–	–	–	0-5
	Portland-silica fume cement	CEM II/A-D	90-94	–	6-10	–	–	–	–	–	–	–	0-5
	Portland-pozzolana cement	CEM II/A-P	80-94	–	–	6-20	–	–	–	–	–	–	0-5
		CEM II/B-P	65-79	–	–	21-35	–	–	–	–	–	–	0-5
		CEM II/A-Q	80-94	–	–	–	6-20	–	–	–	–	–	0-5
		CEM II/B-Q	65-79	–	–	–	21-35	–	–	–	–	–	0-5
	Portland-fly ash cement	CEM II/A-V	80-94	–	–	–	–	6-20	–	–	–	–	0-5
		CEM II/B-V	65-79	–	–	–	–	21-35	–	–	–	–	0-5
		CEM II/A-W	80-94	–	–	–	–	–	6-20	–	–	–	0-5
		CEM II/B-W	65-79	–	–	–	–	–	21-35	–	–	–	0-5
	Portland-burnt shale cement	CEM II/A-T	80-94	–	–	–	–	–	–	6-20	–	–	0-5
		CEM II/B-T	65-79	–	–	–	–	–	–	21-35	–	–	0-5
	Portland-limestone cement	CEM II/A-L	80-94	–	–	–	–	–	–	–	6-20	–	0-5
		CEM II/B-L	65-79	–	–	–	–	–	–	–	21-35	–	0-5
		CEM II/A-LL	80-94	–	–	–	–	–	–	–	–	6-20	0-5
		CEM II/B-LL	65-79	–	–	–	–	–	–	–	–	21-35	0-5
	Portland-composite cement[c]	CEM II/A-M	80-88	<—— 12-20 ——>									0-5
		CEM II/B-M	65-79	<—— 21-35 ——>									
CEM III	Blast furnace cement	CEM III/A	35-64	36-65	–	–	–	–	–	–	–	–	0-5
		CEM III/B	20-34	66-80	–	–	–	–	–	–	–	–	0-5
		CEM III/C	5-19	81-95	–	–	–	–	–	–	–	–	0-5
CEM IV	Pozzolanic cement[c]	CEM IV/A	65-89	–	<—— 11-35 ——>					–	–	–	0-5
		CEM IV/B	45-64	–	<—— 36-55 ——>					–	–	–	0-5
CEM V	Composite cement[c]	CEM V/A	40-64	18-30	–	<—— 18-30 ——>			–	–	–	–	0-5
		CEM V/B	20-38	31-49	–	<—— 31-49 ——>			–	–	–	–	0-5

a The values in the table refer to the sum of the main and minor additional constituents.

b The proportion of silica fume is limited to 10 %.

c In Portland-composite cements CEM II/A-M and CEM II/B-M, in pozzolanic cements CEM IV/A and CEM IV/B and in composite cements CEM V/A and CEM V/B the main constituents other than clinker shall be declared by designation of the cement (for examples, see Clause 8).

Figure 5 The 27 products in the family of common cements (EN 197-1)

Following the requirement in the BS EN 14227-2014 the mixture, with a nominal aggregate size >8 to 31.5 mm, has to content minimum 3% by mass of cement.

Water is another key ingredient, which when mixed with cement, forms a paste that binds the aggregates together.
The water needs to be pure in order to prevent side reaction from occurring which may weaken the concrete or otherwise interfere with the hydration process.

In accordance with Series 800, Clause 880, the mixture design procedure shall determine the properties of the CBGM at a minimum of 3 values of binder contents and a minimum of 2 values of water content for each value of binder content.

This it is need to avoid excessive water loss/drying out of the samples during curing. Once wrapped the specimens have to store in air at the selected curing temperature for a period of time. Two curing procedures are selected:

- Standard curing at 20°C for 28 days: conservative approximation to simulate actual performance of CBGM on site.

  - Standard curing at 20°C for 14 days in a sealed condition and then removed from their mould and immersed in aerated water for 14 days at the same test temperature. This procedure is needed for calculating the loss strength after immersion.

After this period, all specimens are unwrapped, conditioned and tested. The tests carried out to characterise the material are the following:

- **Particle size distribution (PSD)**: the PSD analysis of aggregates involves determining the percentage by mass of particles within different size ranges determined by the method of sieving shown in Figure 6.

Figure 6 Test sieves

The particle size distribution of aggregate is presented as a curve on a semi logarithmic plot, the ordinates being the percentage by mass of particles smaller than the size given by the abscissa (Craig, 2005) Below, in Figure 7, an example of how to represent the PSD results is shown.

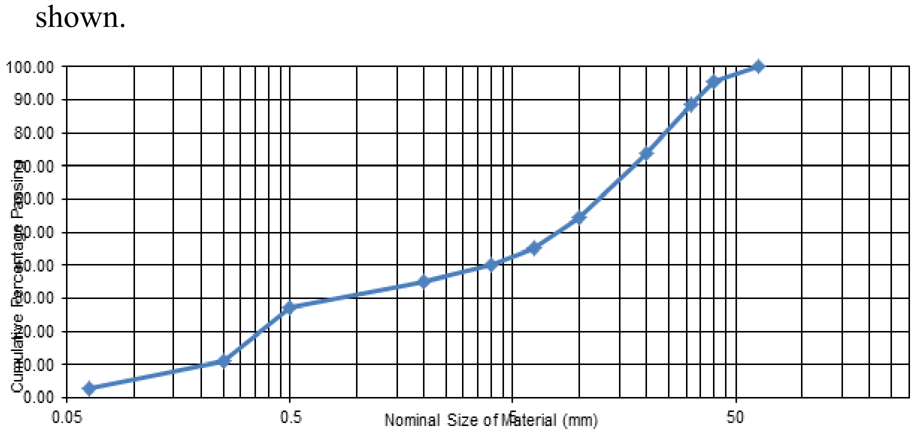


Figure 7 Example of grading curve

- **Water content:** it is a guide for the classification of natural aggregates and is measured on samples used for most field and laboratory tests. This is the most important property of the aggregates, when the density increase, the contact points and interlocking between particle sizes also increases. Because of what the relationship between shear strength and compressibility, it is important to know the quantity of water in aggregates and compare it with a default values that represents a limit of the behaviour (Atterberg Limit). Water content traditionally has been expressed as the ratio of the mass of water present in a sample to the mass of the sample after it has been dried to constant weight, or as the volume of the water in the sample is needed (Nikolaides, 2015).

- **Quality of fine:** It can have a direct effect on the durability, the size fraction less than 0.425 mm is to be non-plastic. This requirement mitigates some of the issues associated with high plasticity clays. High plasticity clays are prone to shrink and swell in response to varying water content; an increase in water content may induce volume change sufficient to break bonds within the HBM. Clay can also coat the surface of the aggregate particles, which results in poor bonding between the particles, and ultimately, poor performance (Järvenpää, 2001).
  Changing the content of water in the aggregates, it is evident that the condition of the material passes from solid to semisolid, after passing from plastic to liquid. The aggregate deformation is strictly related with these continuous variations of conditions. This test provides two individual pieces of information**:**

1. **Plastic Limit** is the dry limit of the clay at which it will crack and fail (from semisolid to plastic)
2. **Liquid limit** is the point at which the clay will change from solid to liquid. When these two values are plotted onto the graph the Plasticity Index can be determined, this gives a reading that determines the type of clay.

The liquid limit is determined by the cone penetration method. It consist of pushing the cone into the ground (the material retained on the 425 μm test sieve has been removed) at a standard velocity of 1 to 2 cm/s while keeping the sleeve stationary.
By recording the sinking of the cone into the cup filled with the material the liquid index is calculated (R. Tanzen, 2016).

The plastic limit is determined by the rolling thread method. In this method, a mass of soil (view Figure 8) is rolled into a thread by hand with a sufficient pressure and at specified rate.

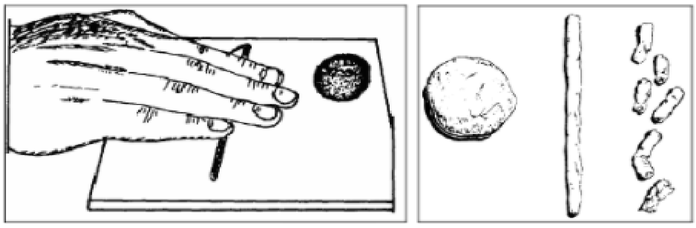


Figure 8 The crumbling thread of traditional plastic limit test (Hashim)

The moisture content, expressed as a percentage of the weight of oven dry soil, at which the soil mass will just begin to crumble when rolled into a thread of about 3 mm, is considered as a plastic limit (R. Tanzen, 2016).

- **Los Angeles test:** in order to produce a high-quality material aggregate abrasion characteristics are important because the constituent aggregates must resist crushing, degradation and disintegration. Los Angeles test (LA) gives a measure of degradation of mineral aggregates. It takes into account degradation due to abrasion or attrition, impact, and grinding. LA abrasion test is a common test method used to indicate aggregate toughness and abrasion characteristics. The material is putted in a cylindrical machine with 11 spherical balls. After 500 revolutions, the material from the tray is retained on 1.6 mm sieve and after is weight. The difference between this weight and the original weight as a percentage express the L.A. abrasion loss value (Collis, 1993).
  Following the requirement in a Series 800, the resistance to fragmentation of coarse aggregates shall have a value of LA_{50} or LA_{60}.
- **Acid soluble sulphate and sulphate content:** as just discussed, CBMG is produced with cement. It is possible that the presence of the any substances in the aggregate in the form of salts or acid can produce a reaction among cement compounds producing destructive expansion, cracking and material detachment. There most important types are sulphur and sulphate attacks. The sulphur does not produce disruptive action but it can transform into acids that can cause aggression to the mix. The sulphur's attack is rarer than sulphate's attack. Chemical reactions with other present substances in the soil can happen, to a greater or lesser extent, on the basis of the condition of the aggregates. They can encourage the entry of oxygen and moisture causing difficult prevision of the damage depending on the sulphur content. The sulphate attack occurs due to interaction between ion SO_4^{--} and other compounds existing within the mix: $Ca(OH)_2$ (Hydrated lime), C-S-H (Hydrate calcium-silicate in charge of hardening) and C-A-H (Hydrate calcium aluminate in charge of setting). For these reasons, the effects of sulphur and sulphate need to be considered carefully providing the durability of CBGM B. In particular the

Acid-soluble sulphate content shall be less than 0.2% and the total sulphur content < 1% (WRAP, 2005).

- **Optimum moisture content:** compaction is the process of increasing the density of the mixture by packing the particles closer together with a reduction in the volume of air; there is no significant change in the volume of water in the mixture. In general, the higher the degree of compaction the higher will be the shear strength and the lower the compressibility of the mixture. An engineered fill is one in which the aggregate has been selected, placed and compacted to an appropriate specification with the object of achieving a engineering performance, generally based on past experience. The aim is to ensure that the resulting fill possesses properties that are adequate for the function of the fill.
  The most common laboratory tests used to obtain the optimum moisture content is the Proctor test.
  This test consists of compaction the specimens with a specific numbers of blows for layers using a rammer (Nikolaides, 2015).
  The specification suggest to making a minimum of five specimens increasing the moisture content.
  The quantity of sample needed depends on the grading as specifies Table 3:

Table 3 Summary of sample preparation methods (EN 13286-2)

Percentage passing test sieves			Mass of sample kg	Proctor mould
16 mm	31,5 mm	63 mm		
100	-	-	15	A
			40	B
75 to 100	100	-	40	B
<75	75 to 100	100	40	B
-	<75	75 to 100	200	C

In addition, for each Proctor mould, the specification recommends the combination of mould sizes, applicable rammer mass and a layer’s number.
Determining for each proctor compaction test the dry density and the water content, the set of results are plotted. From this curve, in correspondence of the maximum dry density obtained it is possible to read the optimum water content like show Figure 9.

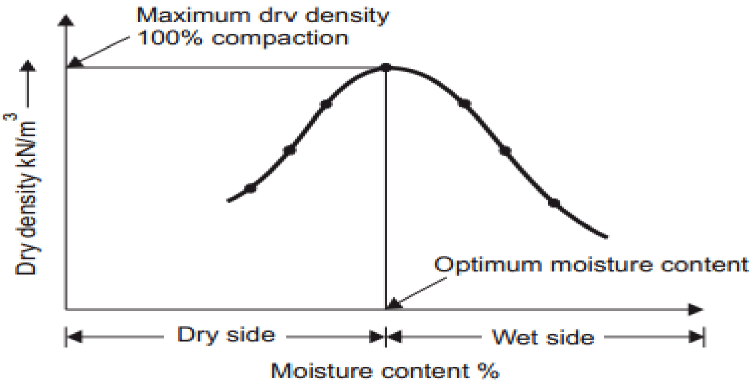


Figure 9 Example of the result graph

Insufficient durability (of the mixture or its component parts) can result in unacceptable degradation of the HBM, chemical attack of the mixture, and/or disruptive volumetric changes. The durability of an HBM can be specified by minimum aggregate requirements (threshold values) and/or by considering the end performance of the mixture, either by durability testing directly or by assessing mechanical performance against available empirical guidance. The mechanical performance classification system for HBMs contained within Series 800 [MCHW1, 2005] and European Standards [BS EN 13286 41 to 43, 2003] can be divided into two systems:

- System one – Based on an indirect method such as compressive strength, R_c; and
- System two – Based on a more fundamental combination of tensile strength, R_t, and modulus of elasticity, E_c.

A compressive strength value (System one classification) is often used as the basis for generic guidance. Care should be taken in relating strength gains, mixture design, material and application specific properties to the use of any generic guidance values, as the overall performance of an HBM is dependent on many factors which cannot be accounted for in such generic values.

- **Compressive strength:** A compressive strength value (System one classification) is often used as the basis for generic guidance. Care should be taken in relating strength gains, mixture design, material and application specific properties to the use of any generic guidance values, as the overall performance of an HBM is dependent on many factors which cannot be accounted for in such generic values. The sample (of a 1:1 height to diameter ratio) is generally placed in between two plates that distribute the applied load across the entire surface area of two opposite faces of the test sample and then the plates are pushed together by a universal test machine causing the sample to flatten (WRAP, 2005). A compressed sample is usually shortened in the

direction of the applied forces and expands in the direction perpendicular to the force. The compressive strength is calculated from Equation 1.
The failure load F is divided by the cross-sectional area resisting the load A_c and it is reported in units of pound-force per square inch (psi) in US Customary units or mega Pascal (MPa) in SI unit:

$$R_c = \frac{F}{A_c}$$

Equation 1 Compressive strength

Compressive strength of specimens depends on many factors such as water-cement ratio, cement strength, quality of concrete material, and quality control during production of concrete.
In Figure 10 the different modes of failure are shown. If the failure of the specimen is like the first four modes, the failure is satisfactory. The others are unsatisfactory.

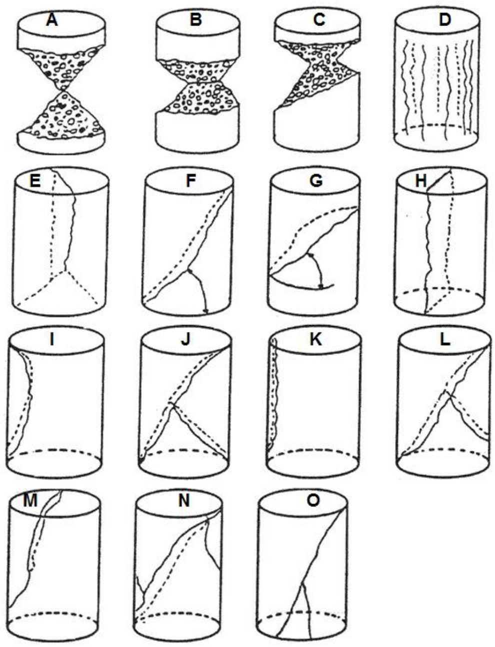


Figure 10 Examples of satisfactory or unsatisfactory failure of cylinder specimen

It is important calculate the retain strength after immersion. The specimens are tested after 14 days curing period in air and another 14 days in air condition.
By partially curing the specimens in water, the test seeks to determine the damage to the specimen that results from expansive reactions. These reactions are promoted by excess of water that moves into the specimen during the immersion period, simplistically mimicking the ingress of water which is typically associated with some common pavement failure mechanisms.

- **Indirect tensile strength:** The indirect tensile test involves loading a cylindrical specimen with compressive loads which act parallel to and along the vertical diametrical plane, as shown in Figure 11.

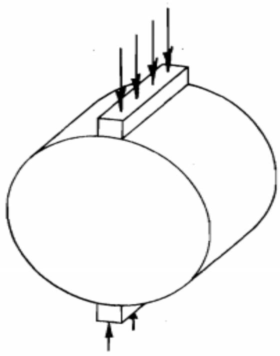


Figure 11 Cylindrical specimen with compressive load being applied

To distribute the load and maintain a constant loading area, the compressive load is applied through a ha1f-inch-wide stainless steel loading strip which is curved at the interface with the specimen and has a radius equal to that of the specimen. This loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametrical plane, which ultimately causes the specimen to fail by splitting or rupturing along the vertical diameter (Figure 12) (Ronald H., 1968) (Nikolaides, 2015).

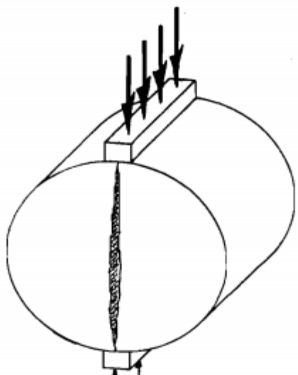


Figure 12 Specimen failing under compressive load

The basic testing apparatus includes loading equipment capable of applying compressive loads at a controlled deformation rate and a bear loading strip, which is used to apply and distribute the load uniformly along the entire length of the specimen.

- **Modulus of Elasticity:** this parameter is used to determine the "flexibility" of the material. This term means the capacity of the material

to return to its original shape or size immediately after stretch or squeeze. This value can be calculated using different tests like the follow:
- The test on compressive strength;
- The direct tensile test; or
- The indirect tensile test.

In this work was used the compressive test method. The reasons for this choose were the simplicity in terms of set up and analysis and sample manufacturing procedures.

The specimen is fitted with a collar that measures the amount of specimen compression, in millimetres, during the application of the load to the top of the specimen.

The results are expressed in Megapascals (MPa).

This testing programme used a height to diameter ratio of 2:1, as this meant that the compressive strength test result could more readily be used to characterise the mixture. Strain ε is measured over the central part of the cylindrical specimen and the strain at 30% of the peak force F_r is derived. The mixture's stiffness is calculated using Equation 2

$$E_c = \frac{1.2\ F_r}{\pi D 2 \varepsilon_3}$$

Equation 2 Modulus of Elasticity

Where:
E_c is the modulus of elasticity in compression (MPa)
F_r is the peak force (N)
D is the specimen diameter (mm)
ε_3 is the longitudinal strain of the specimen (at 30% of F_r)

The longitudinal strain ε_3 was measured with strain gauges on the wall of the specimen.

Three straight lines offset 120° were fixed in the central part of the specimen. Following the specification the central part shall be at least four times the maximum dimension of the aggregate.

The machine test apparatus required is formed by three strain gauges each fixed to the wall of the specimen by four screws and two elastic cords.

Three transducers are fixed to the gauges to determine the displacement between the top and the bottom screws.

The strain ε3 is equal to (Equation 3):

$$\varepsilon_3 = \frac{\Delta l}{l_0}$$

Equation 3 Longitudinal strain

-Δl is the average of the three measurements;

-l_0 is the length of the central part of the specimen

The resulting modulus of elasticity measurement is sometimes referred to as a static stiffness due to the slow loading rate of the test. The test loading rate is specified as a continuous and uniform loading so that rapture occurred within 30 to 60 seconds of commencement.

- **Immediate bearing index:** The immediate bearing index test is an empirical measure of the resistance to penetration. The relationship between force and penetration is determined when a cylindrical piston of a standard cross-sectional area (49.53 mm dimeter) is made to penetrate a specimen of a mixture, contained within a mould, at a given rate. The specimen is compacted using either Proctor effort in accordance with EN 13286-2. The immediate bearing index (IBI) is calculated by expressing the force on the piston for a given penetration as a percentage of a reference force. The purpose is to apply a seating force to the piston and to record the load reading at penetration increments of 0.5 mm up to a total penetration not exceeding 10 mm. The IBI was determined no later than 90 minutes after mixing. The expression of results consists on a graph with a force as ordinate and the corresponding penetration as abscissa as shown in Figure 13.

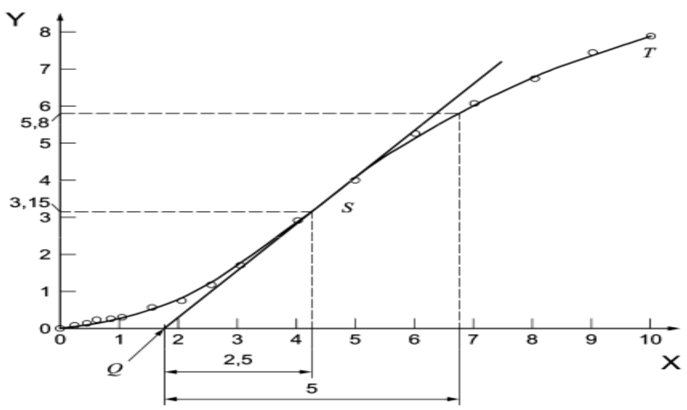


Figure 13 Force/Penetration curve

The IBI is expressed as the ratio of the load resistance (test load) of a given soil sample to the standard load at 2.5mm or 5mm penetration, expressed in percentage:

CBR = (Test load/Standard load) × 100

The standard load for 2.5mm and 5mm penetrations are 1370 kg and 2055 kg respectively. The higher percentage is the immediate bearing index.

1.6 PAVEMENT CLASSIFICATION

According to the constituent materials, there are three different types of pavements (Thom, 2008):

- Flexible
- Rigid
- Composite

Flexible Pavement

Flexible pavements comprise layers of natural granular material covered with one or more bituminous layers, and as the name imply, are considered to be flexible. Flexible pavements will transmit wheel load stresses to the lower layers by grain-to-grain transfer through the points of contact in the granular structure.
It will flex (bend) under the load of a tyre. The wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth.
In flexible pavements, the load distribution pattern changes from one layer to another, because the strength of each layer is different. The common practice is to use the strongest material (least flexible) in the top layer and the weakest material (most flexible) in the lowest layer. The reason for this is that at the surface the wheel load is applied to a small area, resulting in high stress levels. Deeper down in the pavement, the wheel load is applied to larger areas, and the result is lower stress levels, enabling the use of weaker materials (Bridges, 1997).
In Figure 14 is shown the typical cross section.

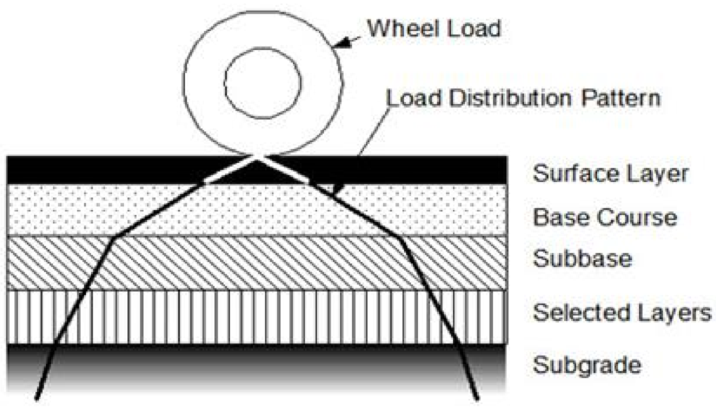


Figure 14 Flexible Pavement (SANRAL)

The *surface course* is the layer directly in contact with traffic loads. It comprises high-quality asphalt; therefore, it is an expensive layer, strong enough to resist the distortion under traffic and designed to provide skid resistance.
As already stated, this layer will prevent the entrance of excessive quantities of water into the underlying base, sub-base and sub-grade.
It is made up of a mixture of various selected aggregates bound together with asphalt cement or other bituminous binders.

The *base* is the layer that gives the pavement most of its strength; hence, its mechanical properties should be carefully chosen to extend the pavement life. It may be built with asphalt, hydraulically bound or granular materials. It is a relatively thick layer; therefore, it should be as cheap as possible, without compromising the mechanical performance. With these constraints, it is common practice to use mixtures with large aggregates sizes.
The materials composing the base course are select hard and durable aggregates, which generally fall into two main classes: stabilized and granular. The stabilized bases normally consist of crushed or uncrushed aggregate bound with a stabilizer, such as Portland cement or bitumen.

The *sub-base* course is important to improve drainage, provide a structural support. The subbase course functions like the base course. The material requirements for the subbase are not as strict as those for the base course since the subbase is subjected to lower load stresses. The subbase consists of stabilized or properly compacted granular material. When this layer is subjected to high stresses the performance of an asphalt or concrete base is dependent on the stiffness of this layer. For this reason, it is made with high-quality granular materials or hydraulically-bound layers (Edward , 2007).

The last layer is the *sub-grade*. This layer of natural (or imported) soil is prepared to receive the stresses from the layers above. It should be compacted to the desirable density, near the optimum moisture content.

Rigid pavement

Rigid pavement is constructed of from cement concrete or reinforced concrete slabs. Figure 15 shows a typical cross section of a rigid pavement.

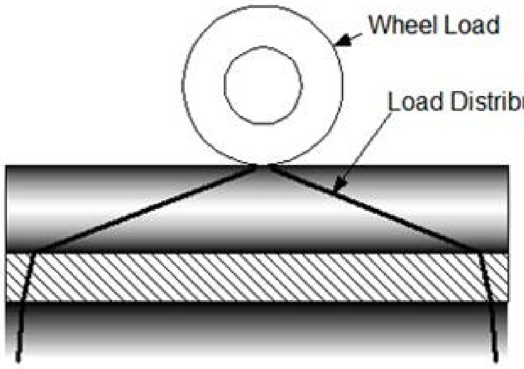


Figure 15 Rigid Pavement (SANRAL)

The structural capacity of rigid pavements is only dependent on the characteristics of concrete slab.
Because of its rigidity and high modulus of elasticity, it tends to distribute the load over a relatively wide area of soil as show the load distribution pattern in Figure 15. Its structural strength is provided by the pavement slab itself by its beam action.
It is laid by a paving machine, often on a supporting layer that prevents the pressure caused by traffic from pumping water and natural formation material to the surface through joints and cracks. Concrete shrinks as it hardens, and this shrinkage is resisted by friction from the underlying layer, causing cracks to appear in the concrete. Cracking is usually controlled by adding steel reinforcement in order to enhance the tensile strength of the pavement and ensure that any cracking is fine and uniformly distributed. Transverse joints are sometimes also used for this purpose. Longitudinal joints are used at the edge of the construction run when the whole carriageway cannot be cast in one pass of the paving machine .
In contrast to flexible pavements, rigid pavements are placed on the prepared sub-grade or on a single layer of granular or stabilized material. (KAMAL, 1992)

Composite pavement

There are several types of composite pavement structures; a composite structure is defined as a multi-layer structure where there is a flexible layer (top-most layer) over a rigid layer.
A composite pavement structure is a structure comprising two or more layers that combine different characteristics and that act as one composite material (Smith, 1963). The two most commonly used materials in a composite structure are a flexible layer (e.g., Hot Mix Asphalt) and a rigid layer (e.g., Plain Cement Concrete, cement-treated base [CTB], cement stabilized base [CSB], rolled-compacted concrete [RCC], or lean mix concrete).

There are different ways of construction of the composite pavements (Figure 16) because an HMA overlay on a CTB can be considered a composite pavement; likewise, a thin PCC overlay on an HMA layer, known as white topping, has also been considered a composite pavement. Furthermore, a PCC surface layer applied on top of another PCC layer before the bottom layer has set may be considered a composite "wet on wet" pavement (Gerardo, 2008).

Figure 16 Composite Pavement (Gerardo, 2008)

In Figure 17 there are two examples of composite pavements are presented where HBM is used for base and foundation layers. In these are represented:
a) Surface layer (asphalt)
b) Binder layer (asphalt)
c) Base layer (HBM)
d) Base layer (Bitumen)
e) Foundation layer (mixture of aggregates)
f) Foundation layer (HBM)
g) Subgrade

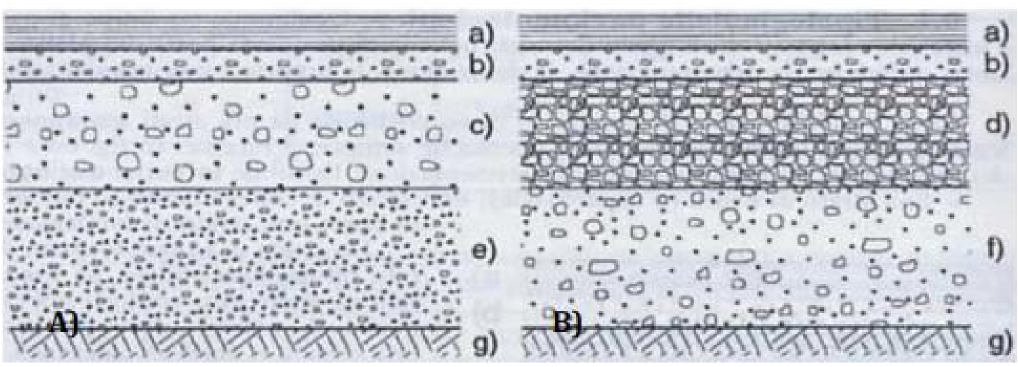


Figure 17 Example of composite pavement

2. METHODOLOGY

As already stated, the aim of this thesis is to investigate the adequacy of using a site-won gravel as aggregates of an HBM for its application as a base course.

The study has different stages:

1. Aggregates characterisation;
2. Mixture manufacture;
3. Analysis of mechanical properties to choose optimum mix design.
4. Pavement design with materials properties obtained in previous phases

2.1 MATERIAL

In order to provide a representative sample, aggregates were taken from 3 different stockpiles at different points and at different heights, in accordance with BS EN 932-1. Four samples were taken from the first stockpile and three samples from the second and third. Each sample taken by the bucket loader was placed in smaller stockpiles in a safe place for sampling by the AECOM technician in accordance with BS EN 932-1. A total of 10 samples were recovered from site from different locations to enable material variability to be assessed.

In Figure 18 and Figure 19 sampling equipment used stockpile 1 are presented. A total of 33 bags (of approximately 25kg each) were collected to provide suitable quantities of material for specimens' production and laboratory testing.

Figure 18 Sampling equipment

Figure 19 Stockpile of material

2.2 METHODS

2.2.1 Particle Size Distribution

As just discussed in a literature review, this test was carried out in order to obtain what sizes (particle size) of particles are present in what proportions (relative particle amount as a percentage where the total amount of particles is 100 %) in the sample particle group to be measured.

PSD was carried out in accordance with the EN 933-1 and the apparatus used is shown in a Figure 20 below:

Figure 20 Apparatus for the grading

Before starting the procedure the material was washed and dried in the oven at 105-110 °C degree to a constant mass to minimise segregation and loss of fines.
The grading was carried out for 7 different samples for the same material to ensure the correct composition.

A sample was passed through a series of standard test sieves having successively smaller mesh sizes.
The sieve sizes used are shown in Table 4:

Table 4 Grading sieves

Size of sieves [mm]
40
31.5
25
20
10
4
2
0.5
0.25
0.063

The mass of soil retained in each sieve is determined and the cumulative percentage by mass passing each sieve is calculated.

As just discussed in the last chapter, the envelope curve of the aggregates shall be contained within the limit curves suggested of the Specification.

2.2.2 Water content

The water content was undertaken following the EN 1097-5. To determine it, the weighed test portion was placed in a clean and dry container. After placing the tray in the oven at 110±5 °C the constant mass was achieved. The water content was calculated in accordance with the following Equation 4:

$$\mathrm{w} = \frac{M_1 - M_3}{M_3} * 100$$

Equation 4 Water content

Where:

M_1 is the mass of the test portion, in grams:
M_3 is the constant mass of the dried test portion, in grams.

2.2.3 Quality of fines

The liquid and plastic indexes were calculated in accordance with EN 13722-2.

The liquid index was determined conducting the cone penetrometer method.
The British fall cone apparatus shown in Figure 21 with a 30° cone and weighing 0.785 N was used during the experimental investigation in order to determine the liquid limit.

Figure 21 Fall-cone penetration apparatus

First of all, it was necessary to take a sample of 20 g in mass from the aggregates which passing 425 μm sieve and mix it with distilled water using spatulas.
The second step was pushed a portion of the mix soil into the cup and strike off excess soil to give a flat level surface.
After that, the tip of the cone was locked when it touched the surface of the soil.
The last step was to fall free the cone for a period of 5 seconds and to record the penetration. Figure 22 shows the steps.
This procedure was required for a different water contents and the liquid limit was read from the plot as the water content on the liquid state line corresponding to a penetration of 20 mm.

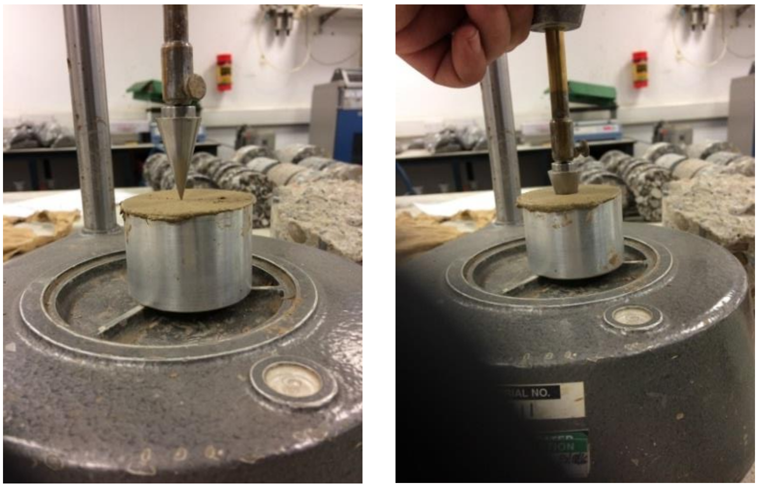


Figure 22 Before and after the penetration

As just discussed in the chapter of the methodology, about 25 g mass of soil shown in Figure 23 was rolled between the palm of the hands until the sample appear to crack on its surface.

Figure 23 Sample of the soil which passing at 425 μm

In a Figure 24 below are shown threads formed following this procedure and the water content measured at this state was the plastic limit.

Figure 24 Threads obtained

2.2.4 Los Angeles Test

This test is carried out in accordance with EN 1097-2.
The used aggregates were passed the 14 mm and retained on the 10 mm test sieve.
In addition, the requirement was that the grading of the tested aggregates has to have a range of 60% and 70% passing the 12.5 mm test sieve.
A sample mass of 5000 g is placed inside a rotating steel drum containing 11 steel spheres or "charge".

As the drum rotates, a shelf inside the drum picks up the aggregate and steel spheres. The shelf carries them around until they drop on the opposite side of the drum, subjecting the aggregate to impact and crushing. Then, the aggregate is subjected to abrasion and grinding as the drum continues to rotate until the shelf picks up the contents, and the process is repeated. The Figure 25 below shows the aggregates obtained after the procedure.

Figure 25 Aggregates after test

Afterward, the aggregate is removed from the drum and sieved on a 1.6 mm sieve. The aggregate retained on this sieve was weighed and the difference between this weight and the original weight was expressed as a percentage and reported as the L.A. abrasion loss value.

Figure 26 shows the equipment used in the L.A. abrasion test

Figure 26 Machine and steel equipment used in the L.A. abrasion test

2.2.5 Acid soluble sulphate and total sulphur content

This test was carried out to other laboratory due to lack of apparatus.

2.2.5 Optimum moisture content

This test was carried out following the EN 13286-2 in order to estimate the mixture density that can be achieved on construction sites and to provide a reference parameter for assessing the density of the compacted layer of the mixture.
In this case the proctor test was undertaken with Proctor mould type B following the requirements displayed in Table – in the literature review section.
The combination of mould size and rammer size were permitted from the specification. In this case the parameters adopted are shown in Table 5:

Table 5 Combination adopted

Type of test	Characteristics of test	Dimension	Proctor mould B
Proctor test	Mass of rammer	Kg	2,5
	Diameter of rammer	mm	50
	Height of fall	mm	305
	Number of layers	-	3
	Number of blows per layer	-	56

Six samples of 6 kg each was produced with the material passing the 31.5 mm.
For the oversize material (retained on 31.5 mm), the particle density was determined in order to ensure the correct calculation of the optimum moisture content requested by the specification.

These samples were mixed with different amount of water. The procedure explained in the BS suggested with gravely mixture a water content of 4% to 6% with an increment of 1% to 2%. In this case study was used the followed percentages of water:

- 3%
- 4%
- 4,5%
- 5%
- 5,5%
- 6%

After preparing the mixture and putting it in the mould, the procedure of compaction with three different layers was carried out with the specific apparatus, see Figure 27.

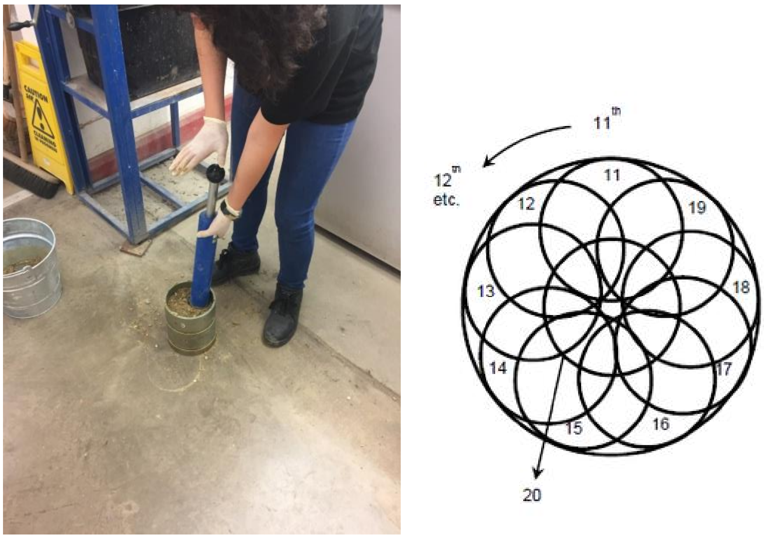


Figure 27 Compaction procedure with recommended patterns

After compaction the extension was taken off and the excess mixture was stroked off as shown in Figure 28.

Figure 28 Sample after rammer compaction

The last step was to remove the sample from the mould as shown in Figure 29 and for calculating the water content was taken the soil sample from the top, middle and bottom portions.

Figure 29 Specimen's extraction

Based on the whole set of results, a curve is plotted for the dry unit weight (or density) as a function of the water content. From this curve, the optimum water content to reach the maximum dry density can be obtained.

2.2.6 Mix Design

As discussed in a literature review, to ascertain the optimum mix design, the mechanical properties of 6 mixes were evaluated. These 6 mixes are manufactured with 3 values of binder content and 2 values of water content.
The cement used to produce CBGM B was CEM II highlighted in the last figure.
The CBGM B was produce in the laboratory using a mixer machine as shown in Figure 30.

Figure 30 Mixer machine

The curing was undertaken because the mixtures need time to gain strength. In order to avoid excessive loss of water the samples during curing are wrapped. The specimens were stored in air at the selected curing temperature for a period. Two curing procedures were selected:

- Standard curing at 20°C for 28 days: conservative approximation to simulate actual performance of CBGM on site;

- Standard curing at 20°C for 14 days in a sealed condition and then removed from their mould and immersed in aerated water for 14 days at the same test temperature.

After this period, all specimens were conditioned and tested as described in the next section.

2.2.7 Compressive strength

The BS EN 13286-41 has given the test instructions.
This test was carried out in order to evaluate the compressive strength before and after immersion. The strength after immersion evaluates the volumetric stability of a HBM. For each mixture, three specimens were cured for 28 days while another three were cured for 14 days in air and another 14 days in water before to be tested.
The machine used is shown in Figure 31 below:

Figure 31 Compressive strength machine

The specimen was centred on the lower platen and at the moment of contact between the upper platen and the specimen the spherical seating was adjusted to achieve uniform contact.
The load was applied in a continuous and uniform manner so that rupture was occurred within 30 s to 60 s of commencement of loading.
The test was completed when the specimen was reached to failure as shows Figure 32.

Figure 32 Failure's phase

2.2.7 Indirect tensile strength

This test was carried out following the EN 13286-42.
It is preceded to the application of the load following the correct positioning of the specimen inside the machine as shown in Figure 33.

Figure 33 Location of specimen

The breakup's condition was achieved due to the progressive increase of the load. It is possible to see the complete break in Figure 34.

Figure 34 Complete break of the specimen C1M1_14

2.2.8 Modulus of elasticity

The EN 13286-43 has given the test instructions.
The mode of testing for the determination of elastic modulus for CBGM B required 28 day accelerated curing of 2:1 (height to diameter) cylindrical specimens.
Specifically were made two specimens for each mixture with 150 mm diameter and 300 mm height. Samples were compacted utilizing a vibrating hammer methodology. Materials were compacted at their optimum moisture content (OMC) as determined but with different cement content. In Figure 35 is shown a specimen after curing period.

Figure 35 Specimen after curing

The longitudinal strain ε was measured with the extensometer on the wall of the specimen.
Three straight lines offset 120° were fixed in the central part of the specimen. Following the specification the central part shall be at least four times the

maximum dimension of the aggregate. In this case was adopted a length of 130 mm having as the maximum aggregate size 31.5 mm.

The machine test apparatus was composed by three strain gauges each fixed to the wall of the specimen by four screws and two elastic cords.

Three transducers were fixed to the gauges to determine the displacement between the top and the bottom screws.

In Figure 36 it is possible to see the positioning of the test specimen in the machine and the apparatus used for this test.

Figure 36 Modulus of elasticity test machine

After connecting the displacement transducers to the data acquisition system the test was carried out until the specimen's breaks as shown Figure 37.

Figure 37 Specimen's breaks

During the test the maximum load at failure was recorded. The displacements between the screws were recorded by the transducers and were sent to the data acquisition system.

Below is reported an example calculation of modulus of elasticity using the results from the compressive strength test.

In Table 6 are reported the values recorded by the data acquisition system:

Table 6 Test results

Time	I transducer	II transducer	III transducer	Δl_1	Δl_2	Δl_3
00:00:00:00	1545.425	1471.951	1441.635	0	0	0
00:00:00:01	1545.425	1471.951	1441.696	0	0	0.061
00:00:00:02	1545.425	1471.951	1441.635	0	0	-0.061
00:00:00:03	1545.49	1472.015	1441.696	0.065	0.064	0.061
00:00:00:04	1545.49	1472.015	1441.696	0	0	0
00:00:00:05	1545.425	1471.951	1441.696	-0.065	-0.064	0
00:00:00:06	1545.425	1472.015	1441.696	0	0.064	0
00:00:00:07	1545.425	1472.015	1441.696	0	0	0
00:00:00:08	1545.425	1472.015	1441.696	0	0	0
00:00:00:09	1545.425	1471.951	1441.696	0	-0.064	0
00:00:00:10	1545.425	1471.951	1441.696	0	0	0
00:00:00:11	1545.425	1471.951	1441.635	0	0	-0.061
00:00:00:12	1545.425	1471.951	1441.635	0	0	0
00:00:00:13	1545.425	1472.015	1441.696	0	0.064	0.061
00:00:00:14	1545.425	1472.015	1441.696	0	0	0
00:00:00:15	1545.425	1472.015	1441.635	0	0	-0.061
00:00:00:16	1545.425	1472.015	1441.696	0	0	0.061
00:00:00:17	1545.425	1472.015	1441.635	0	0	-0.061
00:00:00:18	1545.361	1471.951	1441.635	-0.064	-0.064	0
00:00:00:19	1545.361	1472.015	1441.696	0	0.064	0.061
00:00:00:20	1545.49	1472.015	1441.696	0.129	0	0
00:00:00:21	1545.361	1471.951	1441.696	-0.129	-0.064	0
00:00:00:22	1545.425	1472.015	1441.635	0.064	0.064	-0.061
00:00:00:23	1545.361	1472.015	1441.696	-0.064	0	0.061
00:00:00:24	1545.361	1472.015	1441.696	0	0	0
00:00:00:25	1543.952	1472.015	1441.635	-1.409	0	-0.061
00:00:00:26	1543.888	1472.015	1441.635	-0.064	0	0
00:00:00:27	1543.952	1471.888	1440.96	0.064	-0.127	-0.675
00:00:00:28	1545.233	1471.888	1437.891	0	-0.76	-3.069
00:00:00:29	1543.247	1469.482	1435.989	-1.986	-1.646	-1.902
00:00:00:30	1540.044	1467.456	1441.328	-3.203	-2.026	5.339
00:00:00:31	1521.912	1469.166	1409.967	-18.13	1.71	-31.361

00:00:00:32	1428.147	1434.785	1386.216	-93.76	-34.38	-23.751
00:00:00:33	1352.751	1427.251	1389.837	-75.39	-7.534	3.621
00:00:00:34	1328.336	1426.175	1389.284	-24.41	-1.076	-0.553
00:00:00:35	1326.313	1423.705	1390.328	-2.023	-2.47	1.044
00:00:00:36	1326.439	1423.389	1391.31	0.126	-0.316	0.982
00:00:00:37	1326.818	1423.452	1392.599	0.379	0.063	1.289
00:00:00:38	1327.135	1424.592	1392.844	0.317	1.14	0.245
00:00:00:39	1327.324	1424.781	1392.844	0.189	0.189	0
00:00:00:40	1327.578	1424.718	1392.967	0.254	-0.063	0.123
00:00:00:41	1328.02	1424.592	1392.905	0.442	-0.126	-0.062
00:00:00:42	1328.273	1424.528	1392.905	0.253	-0.064	0
00:00:00:43	1328.526	1424.465	1392.905	0.253	-0.063	0
00:00:00:44	1328.716	1424.402	1392.967	0.19	-0.063	0.062
00:00:00:45	1328.969	1424.402	1392.967	0.253	0	0
00:00:00:46	1329.222	1424.465	1392.967	0.253	0.063	0
00:00:00:47	1329.349	1424.402	1392.967	0.127	-0.063	0
00:00:00:48	1329.538	1424.402	1392.905	0.189	0	-0.062
00:00:00:49	1329.665	1424.402	1392.967	0.127	0	0.062
00:00:00:50	1329.791	1424.402	1392.905	0.126	0	-0.062
00:00:00:51	1329.918	1424.528	1392.905	0.127	0.126	0
00:00:00:52	1330.044	1424.592	1392.905	0.126	0.064	0
00:00:00:53	1330.171	1424.655	1392.905	0.127	0.063	0
00:00:00:54	1330.234	1424.655	1392.599	0.063	0	-0.306
00:00:00:55	1330.361	1424.655	1401.682	0.127	0	9.083
00:00:00:56	1330.487	1424.528	1406.223	0.126	-0.127	4.541
00:00:00:57	1330.55	1424.592	1417.577	0.063	0.064	11.354
00:00:00:58	1330.677	1424.528	-2803.28	0.127	-0.064	-4220.8
00:00:00:59	1332.764	1424.528	-2804.97	2.087	0	-1.684
00:00:01:00	1305.882	1424.465	-2804.47	-26.88	-0.063	0.499
00:00:01:01	1486.97	1424.528	-2803.78	181.08	0.063	0.686
00:00:01:02	-3031.54	1424.592	-2803.97	-4518	0.064	-0.187
00:00:01:03	-3031.48	1424.592	-2804.22	0.066	0	-0.25
00:00:01:04	-3031.28	1424.592	-2803.78	0.199	0	0.437
00:00:01:05	-3032.6	1424.592	-2804.72	-1.325	0	-0.936
00:00:01:06	-3031.81	1424.655	-2804.03	0.795	0.063	0.686
00:00:01:07	-3032.41	1424.845	-2804.41	-0.597	0.19	-0.374
00:00:01:08	-3032.54	1424.971	-2804.78	-0.132	0.126	-0.374
00:00:01:09	-3031.74	1425.035	-2804.03	0.795	0.064	0.748
00:00:01:10	-3032.41	1429.53	-2804.65	-0.663	4.495	-0.623

00:00:01:11	-3032.6	-2785.35	-2804.84	-0.198	-4215	-0.188
00:00:01:12	-3032.27	-2786.35	-2804.53	0.331	-0.998	0.312
00:00:01:13	-3032.6	-2786.28	-2804.84	-0.331	0.063	-0.312
00:00:01:14	-3031.94	-2785.72	-2804.16	0.662	0.561	0.687
00:00:01:15	-3032.27	-2786.16	-2804.47	-0.331	-0.437	-0.312
00:00:01:16	-3032.41	-2786.28	-2804.53	-0.133	-0.124	-0.063
00:00:01:17	-3031.81	-2785.91	-2804.03	0.597	0.374	0.499
00:00:01:18	-3032.74	-2786.35	-2804.9	-0.928	-0.437	-0.873
00:00:01:19	-3031.88	-2785.97	-2804.03	0.862	0.374	0.873
00:00:01:20	-3032.47	-2786.22	-2804.59	-0.597	-0.249	-0.561
00:00:01:21	-3032.07	-2786.16	-2804.34	0.398	0.062	0.25
00:00:01:22	-3031.94	-2786.04	-2804.16	0.132	0.125	0.187
00:00:01:23	-3032.6	-2786.41	-2804.84	-0.662	-0.374	-0.687
00:00:01:24	-3031.74	-2785.97	-2803.97	0.861	0.436	0.874
00:00:01:25	-3032.67	-2786.41	-2804.9	-0.928	-0.436	-0.936
00:00:01:26	-3031.88	-2786.04	-2804.09	0.796	0.374	0.811
00:00:01:27	-3032.27	-2786.22	-2804.47	-0.398	-0.187	-0.374
00:00:01:28	-3032.34	-2786.28	-2804.53	-0.066	-0.062	-0.063

The negative values are referred to a compressive force. Analysing the results it is possible to identify the maximum displacement value which correspond the maximum load at failure. For example, on closer inspection, it is possible to discern that there were a lot of negative values as a result of the great sensibility of the instrument. For this reason the maximum negative displacement has to be found into a range of negative values highlighted in red in Table --. The positive values into this range were considered as noise.

Knowing the time of the peak displacement, the time of the peak force was noted.

The next step was to plot the force-time graph in order to derive the equation of the trend curve and calculate the ε corresponding values. In Figure 38 is shown an example for the only I transducer.

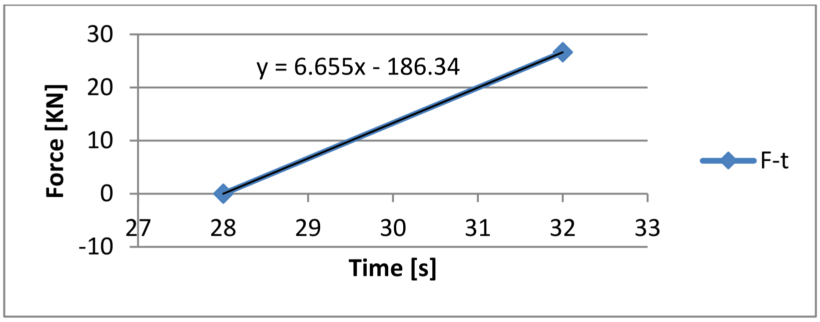


Figure 38 Force-time

The corresponding values were reported in Table 7.

Table 7 Strain values

Time [s]	Force [KN]	$\varepsilon = \Delta l/l_0$
28	0	0
29	6.65	1.52E-05
30	13.31	2.46E-05
31	19.96	1.39E-04
32	26.62	7.21E-04

The last step was to plot these values of the forces and strains (Figure 39) and add a trend line which better represented a typical trend shown in Figure --.

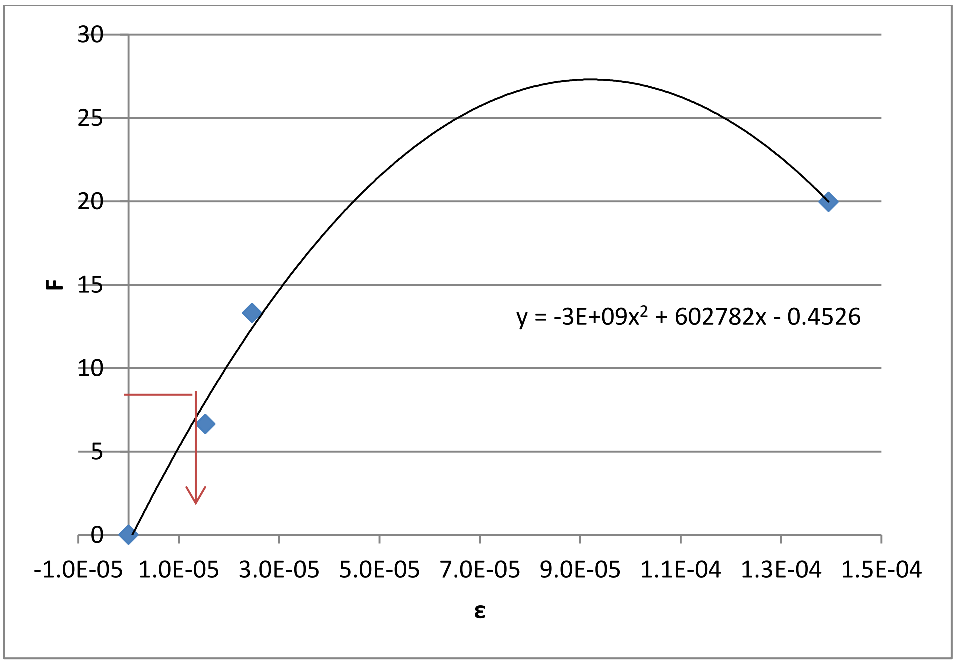


Figure 39 Force-strain

From the trend line's equation was possible to calculate the longitudinal strain of the specimen when $F=0.3F_r$ and finally the Young's Modulus, see Table 8.

Table 8 Results

0.3F [KN]	ε_3	E_c (Mpa)
7.986	1.51E-05	30266

This procedure is valid for the all specimen.

2.2.9 Immediate bearing index

This test was carried out in accordance with EN 13286-47:2012. The testing consists of measuring the compressive load sustained by the specimen as a 152 mm diameter loading piston is driven into the top of the specimen at strain rate of 1.27 mm per minute. Load measurements are reported at every 0.5 mm of penetration.

The mixture was prepared with the same mix machine used in the previous tests as shown Figure 30. Three IBI tests were carried out for mixtures having optimum water content of 4.5% and a lower content of cement of 4%.

After making the mixture, the specimens were prepared following the Proctor compaction effort. Proctor mould B with appropriate spacer disc was used for the specimens manufacture. Each specimen was compacted using 56 blows per lift in three equal lifts by weight with a rammer B of 2.5 kg. The procedure is already explained in the literature review section.

After compaction, the extension collar was removed and the mixture flush with the top of the mould was trimmed with the scraper as shown Figure 40.

Figure 40 Steps of compaction

The mass of the mould without the baseplate was recorded and after repositioned it the specimen was placed into the test's machine to start the test. The machine used is shown in Figure 41.

Figure 41 IBI testing machine

Before to start the test the seating force of 10 N was applied to the piston and the reading of the force measuring device as the initial zero was recorded.

Load and penetration were displayed in real time on the large graphic screen together with the curve as shown in Figure 42.

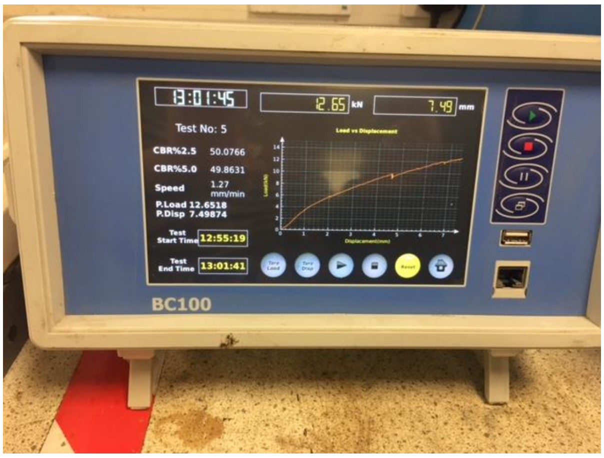


Figure 42 Graph test result

After the testing the dry density of each specimen was estimated from the wet density measured immediately after compaction and the moisture content was measured immediately after testing. From the test curve in Figure 43 it is possible to read off the forces in KN corresponding to the 2.5 mm and 5 mm penetration.

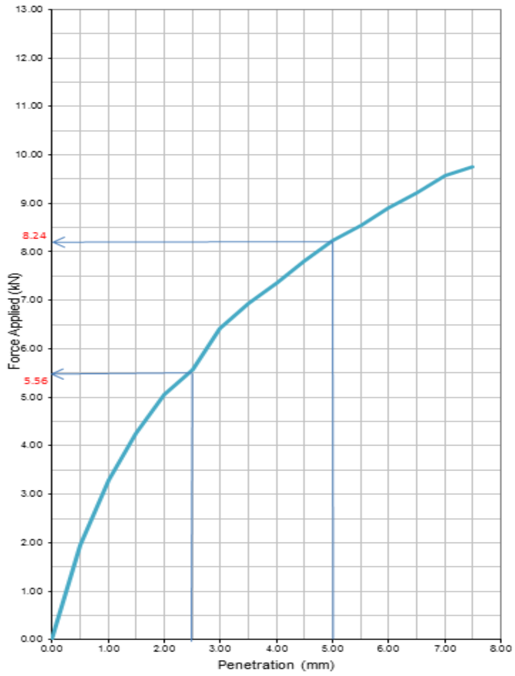


Figure 43 Test result

Using the Equation 5 below CBR was calculated for each penetration:

$$CBR_{2.5mm} = \frac{Test\ load}{Standard\ Load} \times 100 = \frac{5.56\ KN}{13.2\ KN} = 42$$

$$CBR_{5mm} = \frac{Test\ load}{Standard\ Load} \times 100 = \frac{8.24\ KN}{20\ KN} = 41$$

Equation 5 California Bearing Ratio

The higher percentage was taken as Immediate Bearing Index.

3. TEST RESULTS

3.1 Particle Size Distribution

The Particle Size Distribution (PSD) test results are presented in Table 9 and compared to the specification for material type CBGM B, from BS EN 14227-2, CBGM 1, 0/31.5 mm. Ten samples were selected from the 10 different locations to ascertain the uniformity of the stockpiles. For samples 8, 9 and 10, two sieves (1mm and 0.125mm) were added.

Table 9 Summary of PSD results

Sieve Size (mm)	Raw aggregate % Passing										BS EN 14227-2	
	Sample location										Min%. Passing	Max.% Passing
	1	2	3	4	5	6	7	8	9	10		
63	100	100	100	100	100	100	100	100	100	100	-	-
40	80	100	100	100	93	100	96	97	100	97	100	-
31.5	80	95	96	93	89	92	89	97	90	95	85	100
20	67	83	83	82	65	81	74	89	88	82	75	100
10	48	60	63	60	49	65	55	67	65	64	65	94
6.3	39	49	53	47	41	56	45	57	55	53	44	78
4	32	42	47	40	37	50	40	50	48	46	26	61
2	27	37	41	35	33	44	35	43	41	39	18	50
1	-	-	-	-	-	-	-	37	35	33	-	-
0.5	18	26	31	22	23	29	27	25	24	24	8	30
0.25	9	10	14	6	8	8	11	9	8	8	6	22
0.125	-	-	-	-	-	-	-	3	3	4	-	-
0.063	3	4	8	2	2	2	3	2.2	2.1	2.7	3	11

Results are plotted against the specification requirements in Figure 44. It can be observed how the upper and lower part of the grading curve does not meet these requirements.

In order to adjust the grading in accordance with BS EN 14227-2, aggregates over 40mm were removed from the coarse fraction. It has to be taken into account that the grading envelope provided in BS EN 14227-2 includes all the particles present in the mix, i.e. including cement. Therefore, the addition of cement will boost the lower part of the curves, making them fall inside the required envelope.

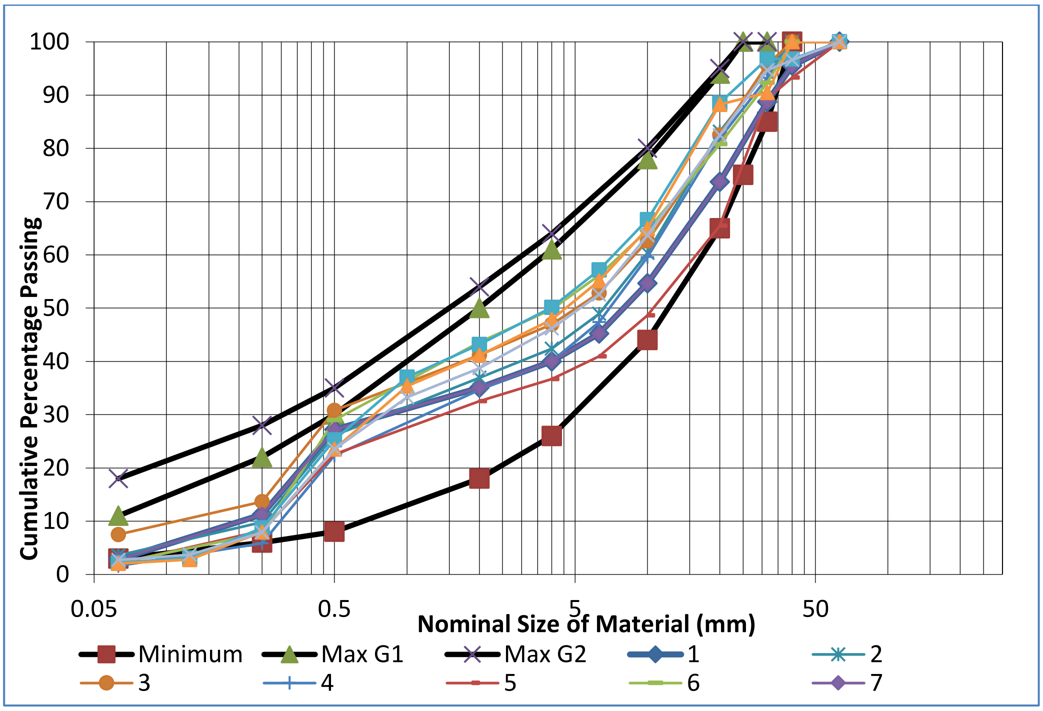


Figure 44 Grading results

There is some variation between sample locations, in particular samples 1 and 5 were found to be coarser compared with other locations. However, with the removal of oversize and addition of cement it is expected that all sample locations will fall within the required grading limits or CBGM B.

3.2 Water content

The Moisture content results (as received from site) are summarised in Table 10. A sample from each location was analysed to ascertain uniformity.

Table 10 Moisture content results

Sample laboratory reference	Sample location	Moisture Content %
0644.01.00	1	5.6
0644.02.00	2	5.3
0644.03.00	3	4.8
0644.04.00	4	4.3
0644.05.00	5	3.1
0644.06.00	6	2.5
0644.07.00	7	4.1
0644.19.00	8	4.2
0644.20.00	9	2.2
0644.21.00	10	2.7

The variability of moisture content is related to the different locations were the samples were taken and the different days.

3.3 Optimum moisture content

The optimum moisture content was calculated in accordance with BS EN 13286-2, using proctor compaction. The results are plotted in Figure 45.

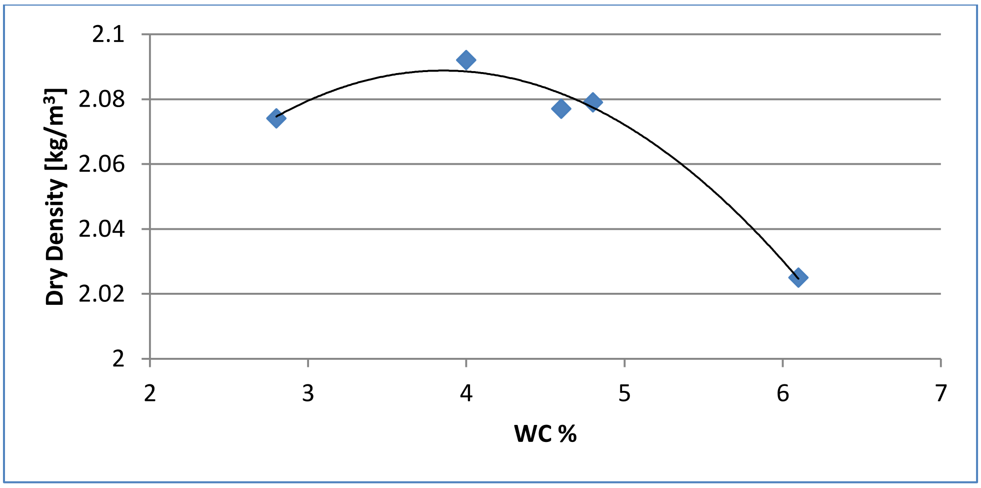


Figure 45 Optimum moisture content

The moisture curve obtained is relatively flat, which suggests that the moisture content does not affect greatly the compaction. From these results it was concluded that the optimum water content should be around 4-4.5%.

With these results it was agreed to manufacture specimens with 4.5 and 6.5% of water to ascertain mechanical properties.

3.4 Aggregates quality tests

As already described in section 1.2, other tests were carried out to ascertain the aggregates quality. In Table 11 these results are summarised.

Table 11 Aggregates quality

Test		**Result**
Los Angeles		LA_{22}
Plasticity index		0-16
Acid soluble sulphate (%)		0.0.1
Total sulphur (%)		<0.01
Water absorption (%)	Fines (4-0.063mm)	1.93

Test		Result
	Coarse (40-4mm)	0.35
Particle density		2.63

The results obtained show a variable material with a plastic index between 0 and 16. Further investigation should be carried out to analyse if the plastic material can be localised and disposed, as a plastic material cannot be used in HBM for base course.

3.5 Mixing procedure

As already stated 3 cement contents and 2 water contents were chosen for the mix design. It was decided to produce the mixtures shown in Table 12.

Table 12 Mixtures manufactured

Mixture name	Cement content (%)	Water content (%)
C1M1	4	4.5
C1M2	4	6.5
C2M1	5	4.5
C2M2	5	6.5
C3M1	6	4.5
C3M2	6	6.5

The mixing procedure is shown in Figure 46. In photograph of the mixer, a sealed specimen is shown in Figure 47.

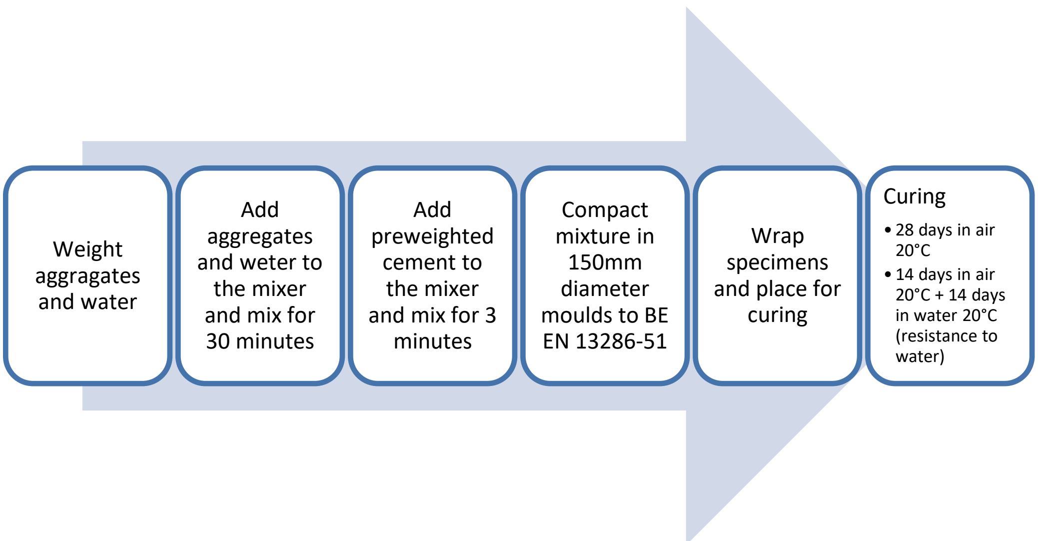
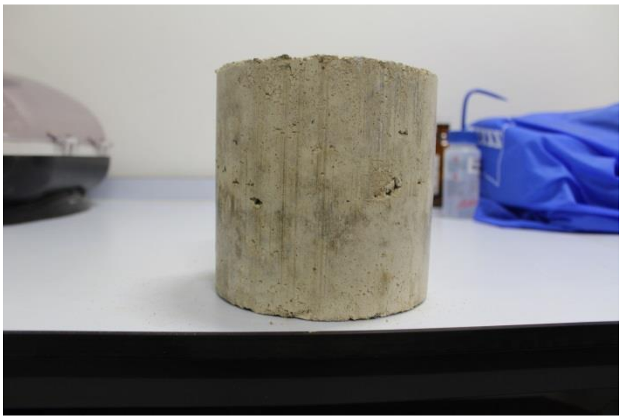


Figure 46 Mixture manufacture procedure

Figure 47 Sealed specimen

After the curing period the specimens were unwrapped and tested as described in the next section.

The samples were removed from the mould using a pump and their looks are shown in a Figure 48 below:



(A)

(B)

(C)

Figure 48 Samples after curing period

It can be seen as not all the specimens have the same aspects.
In this regard, it can be made some consideration:

- The material used for the mix was composed of material with different sieves. As just discussed, the aggregate particles >40 mm were taken off from the soil but still large aggregates were left. For this reason, during the compacting stage, the large aggregates have had a tendency to go outward and the fine material was remained inside. The third photo in Figure 48 shows this aspect. Despite that, after to carry out the tests, the specimens has been demonstrate to be capable to testing for the corrected compaction inside between stone end binder. This is shown in Figure 49.

Figure 49 Good inside compaction

- The good proportion between water and binder has an important role in the mix phase. It is possible that the content of water is not enough to make hydration reactions happen or in the opposite cases the water content exceeds the required amount. These could lead to incorrect ratio water/cement and the subsequence is shown from the pictures in Figure 32. For example Figure 49 (C) represents a sample in which the water content is not enough for the percentage of cement. For this reason, the surface of the specimen is not smooth and homogeneous while happen is Figure 49 (A).

Samples were tested for compressive strength (R_c), static stiffness in compression (E_c) and indirect tensile strength (R_{it}).
A summary of the tests is included in Table 13.

Table 13 Summary of the tests

Laboratory Test Method	Mechanical Property Measured
BS EN 13286 43: Modulus of Elasticity	Secant Stiffness (E_c) based upon 30% peak load
BS EN 13286-41:Compressive Strength	Compressive strength (R_c)
BS EN 13286-42: Indirect tensile Strength	Indirect tensile Strength (R_{it})

The dimensions of each test specimen were determined prior the tests by taking an average of four height measurements and six diameter measurements at equidistant points around the specimens. These measurements are reported in Appendix A.

In the follow treatment will be considered in detailed way exclusively the first relative specimens to every typology of test. The remainders will be analysed in schematic and synthetic way.

3.6 Compressive test

Three specimens for each mixture were tested following curing. The test results presented in Appendix A and summarised in Table 14.

Table 14 Compressive strength results

Sample reference	Density (kg/m^3)	Compressive Strength (MPa)	Main (MPa)	Coefficient of Variation
C1M1_1	2360	3.75		
C1M1_2	2380	2.9	3.28	±0.13
C1M1_3	2290	3.18		
C1M2_7	2360	2.4		
C1M2_8	2320	3.57	3.13	±0.20
C1M2_9	2360	3.42		
C2M1_17	2340	2.26		
C2M1_18	2330	2.75	2.5	±0.14
C2M1_19	2330	5.41*		

Sample reference	Density (kg/m³)	Compressive Strength (MPa)	Main (MPa)	Coefficient of Variation
C2M2_10	2330	2.63		
C2M2_11	2350	1.67	1.99	±0.28
C2M2_12	2330	1.67		
C3M1_27	2320	1.8		
C3M1_28	2290	2.35	2.06	±0.13
C3M1_29	2320	2.03		
C3M2_21	2330	3.13		
C3M2_22	2330	2.81	3.16	±0.11
C3M2_23	2320	3.53		

Notes:

*Unrealistic value, not used for the average

All specimens were broken following a failure type D as shown in Figure 50 so this point was satisfied.

Figure 50 Failure type D

The results show that the optimum cement and moisture content corresponds to 4 (C1) and 4.5% (M1) respectively.

Compressive strengths in the region of C8/10 were envisaged based on past experience with gravel aggregate. Some samples showed visual signs of honeycombing due to coarse aggregate and a lack of fines in localised spots at the surface. However, on inspection of crushed specimens material appeared uniform and well compacted. Furthermore, C1 specimens showed similar density to other specimens and as such compaction is not thought to be a major cause of lower than expected strengths. It is thought that the fines, which have been characterised as 'plastic' in some areas play a major effect in limiting the compressive strength results obtained.

To ascertain the resistance to water, specimens were cured for 14 days in air at 20°C and then immersed in water at 20°C for another 14 days, as specified in clause 880 from series 800 from MCHW. After this curing period, specimens were tested for compressive strength to BS EN 13286-41. This test was carried out on mixtures with the optimum moisture content found from the strength results. Results are presented in Appendix A and summarised in Table 15.

Table 15 Retained strength

Sample reference	Retained strength (%)
C1M1	107.9
C2M1	87.5
C3M1	82.57

Mixture C1M1 has a retained strength greater that 100%. This may suggest that there is still active cement in the mixture that hardens when the specimen is immersed in water.

All the mixtures have greater retained strength than the minimum of 80% specified in clause 822 from series 800 MCHW.

Analysing the retained strength it is possible to make considerations:

- The C1M1 had a bigger value because the strength after immersion is greater than non –immersed strength. As just discussed in the preview chapter, it is really important to have a good ratio water/cement in order to take place the hydration reactions. Probably in this case, the immersion in water has increased the compressive strength giving to the specimen the correct content of water to complete the reactions.

- The C2M2 is the only mixture which complies with the requirement having the retain strength >80%.

- The C3M1 does not comply with the requirement. This mixture has a slow R_c value in relation to other mixtures. In addition, the mixture with the same cement content but with higher water content (C3M2) shows up the strength much higher than C2M1. The reason is the same as previous. The mixture needed more water to increase the resistance. In Figure 51 it can be seen the important role of the ratio water/content in terms of aspect and better compaction.

(A)

(B)

(C)

Figure 51 C3M2 (A), C3M1 immersed in water (B), C3M1 non-immersed (C)

- Determination of peak stress (strength) requires a uniform specimen surface. It was observed that the influence on the quality of test outputs is mitigated by ensuring a high level of workmanship during sample manufacture. In some cases, the surface of the specimen was not completely homogeneous. The excess of pores on the surface have influenced the density and the capacity to absorb water decreasing the strength and stiffness of the material.

- Following the EN 14277-1 mixture shall be classified by compressive strength. The class of compressive strength shall be selected from Figure 52 in combination with the selected method of specimen manufacture.

Column	1	2	3
Line	Minimum R_c for cylinders of slenderness ratio 2[a] MPa	Minimum R_c for cylinders of slenderness ratio 1[a] and cubes MPa	R_c Class
1	0,4	0,5	$C_{0,4/0,5}$
2	0,8	1	$C_{0,8/1}$
3	1,5	2	$C_{1,5/2}$
4	2,3	3	$C_{2,3/3}$
5	3	4	$C_{3/4}$
6	4	5	$C_{4/5}$
7	5	6	$C_{5/6}$
8	6	8	$C_{6/8}$
9	8	10	$C_{8/10}$
10	9	12	$C_{9/12}$
11	12	16	$C_{12/16}$
12	15	20	$C_{15/20}$
13	18	24	$C_{18/24}$
14	21	28	$C_{21/28}$
15	24	32	$C_{24/32}$
16	27	36	$C_{27/36}$
17	30	40	$C_{30/40}$
18	33	44	$C_{33/44}$
19	36	48	$C_{36/48}$
20	Declared value	Declared value	C_{DV}
[a] If cylinders with slenderness ratios other than 1 or 2 are used, then the correlation with cylinders of either slenderness ratio 1 or 2 shall be established before use.			

Figure 52 Compressive strength classification

With a compressive strength value of 3.3 MPa Figure 37 suggested that the R_c class was $C_{2,3/3}$.

The Design Manual for roads and bridges provides the details of permitted materials and of thickness for the construction of pavements. This specification suggests the surfacing thickness for Flexible with HBM Base for a different series of compressive strength class. For a Base layer the minimum R_c class required is C8/10.

This is an important result for this study because it is clear that this material cannot be used as a pavement base layer. The remain material is compliant for use as a sub-base within the pavement foundation.

3.7 Indirect tensile strength

The test of indirect tensile strength consists in the application of a compression diametrical strength to a cylindrical specimen.
The results are presented in Appendix A and summarised in Table 16.

Table 16 ITST resuls

Sample reference	Density (kg/m³)	Tensile strength (MPa)	Main (MPa)	Coefficient of variation
C1M1_13	2340	0.3		
C1M1_14	2330	0.5		
C1M1_15	2340	0.4	0.5	±0.36
C1M1_16	2330	0.7		
C1M1_47	Failed	Failed		
C2M1_20	2310	0.5		
C2M1_35	2330	0.4		
C2M1_36	2320	0.5	0.5	±0.21
C2M1_37	2330	0.4		
C2M1_38	2350	0.5		
C3M1_33	2310	0.6		
C3M1_43	2300	1.0		
C3M1_44	2000	1.2	0.9	±0.26
C3M1_45	2310	0.9		
C3M1_46	2300	0.7		

From Figure 53 it can be seen that the type fracture face is normal and it could be for all of the specimens.

Figure 53 Normal failure

3.8 Modulus of elasticity

The elasticity modulus test was carried out mixes with the optimum water content only. The results are presented in Appendix A and summarised in Table 17.

Table 17 Elasticity Modulus

Sample reference	Elasticity Modulus (MPa)
C1M1	23543
C2M1	41275
C3M1	40440

The failure for all the specimens was the type D as just shown in Figure 54.

Figure 54 Satisfactory failure

3.9 Immediate bearing index

The IBI test was carried out in 3 samples with the optimum moisture and cement content. The Series 800, Clause 880, suggests taking the IBI as the average value for a set of 3 test specimens. The results are presented in Appendix A and summarised in Table 18.

Table 18 Immediate bearing index results

Sample reference	IBI (%)	Mean Value
1	50	46
2	42	
3	29* not used	

Analysing the results it is possible to notice that the CBR values for the first two tests differ greatly from the third. This could be due to different reasons. One reason could be the variability of the method. A wrong compaction might affect the penetration. Another reason could be the composition of the specimen, if two relatively big aggregates happen to be together, this could also affect the compaction.

It was decided to discard the lowest value as it was thought as a procedural error. For this reason, the final value is calculated as the average between the first two tests. Following the EN 14227-1 the mixture with an immediate bearing index less than 40 may not support immediate trafficking and should be used with care. In this case the values comply with the minimum requirement.

4. PAVEMENT DESIGN

Design is a topic that is very hard to tie down. The problem is that there is never a unique solution to any given problem. When planning construction of a major pavement, there will always be a number of different options as to which combinations of materials to use (Thom, 2008).

Pavement structural design is a complex task. The heterogeneous mix of vehicles, axle load and types make the traffic load various. It can vary with time throughout the day, from season to season, and over the pavement design life. There are a lot of factors which affect the pavement materials response as temperature, moisture, stress state and magnitude time and other more. Several developments over recent decades have offered an opportunity for more rational and rigorous pavement design procedures. Most pavement design and maintenance today is carried out with reference to a document (guides with design charts) or a computer program (Thom, 2008).

Before the 1920s, pavement design consisted basically of defining thicknesses of materials in order to provide strength and protection to a soft, weak subgrade. As important as providing subgrade support, it was equally important to evaluate pavement performance through ride quality and other surface distresses that increase the rate of deterioration of pavement structures (Montusci, 2011).

Laboratory test data or test track experiments were developed in order to focus point on performance hereby developing the empirical method.

New design criteria were required to incorporate such failure mechanisms (e.g., fatigue cracking and permanent deformation in the case of asphalt concrete). The Asphalt Institute method and the Shell method are examples of procedures based on asphalt concrete's fatigue cracking and permanent deformation failure modes.
These were the first to use linear-elastic theory of mechanics to compute structural responses (in this case strains) in combination with empirical models to predict number of loads to failure for flexible pavements (Papagiannakis A. T., 2008).

Studies in pavement engineering have shown that the design pavement procedure is either empirical or mechanistic.
An empirical approach is one which is based on the results of experiments or experience. The mechanistic approach involves the determination of material parameters for the analysis, at conditions as close as possible to what they are in the road structure. The mechanistic approach is based on the elastic or viscoelastic representation of the pavement structure (Emmanuel O. Ekwulo, 2009).

4.1 EMPIRICAL METHODS

The empirical method is based on the results of experiment or experience. The relationship between input variables and outcomes are obtained from a large number of observations. Empirical approaches are often used as an expedient when it is too difficult to define theoretically the precise cause and effect relationships of a phenomenon (Huang, 2004).

These methods are based on design charts, from which it is possible to extract the values of the layer's thickness for a determined level of traffic, the type of material and the subgrade strength.

The main limitations of empirical models are:

- Developed for use under particular conditions – difficult to use under different conditions.
-Most of them do not contain material properties.
-They are not comprehensive (do not consider all influencing factors) (Hakim, 2011).

4.2 ANALYTICAL METHODS

The analytical methods designs are based on the fundamental engineering properties of materials in order to calculate layer thickness. These methods consider traffic loading, failure modes and the expected pavement performance. Multi-layer elastic systems, plate on elastic foundation or finite element are used as structural analysis models to calculate stresses and strains which are compared with the allowable values for the material. The aims are to control concrete and asphalt fatigue cracking, pavement deformation and even pavement deterioration mechanism. These design approach allows non-standard materials and construction types (Hakim, 2011).

4.3 MECHANISTIC APPROACH

These methods are based on an analysis of the engineering response to the pavement based on the load applied, or essentially a theoretical analysis of the pavement

Mechanistic models require knowledge of the mechanical behaviour of the various materials as their stress-strain responses.

However, the materials used in pavements do not have linear elastic behaviour and using linear elastic models is a large simplification.

Predictions from any deterministic or mechanistic model only provide the mean value, and are not capable of modelling the dispersion. The use of probabilistic approaches to address such limitations on performance modelling has been suggested elsewhere (Li, 1996)

4.5 MECHANISTIC-EMPIRICAL METHODS

Mechanistic-empirical models combine some mechanical modelling with an empirically calibrated transfer function, relating the distress prediction to some calculated critical stress/strain in the pavement (Dalla Valle, 2015).
It is considered as a hybrid approach. Empirical models are used to fill in the gaps that exist between the theory of mechanics and the performance of pavement structures. Simple mechanistic responses are easy to compute with assumptions and simplifications (i.e., homogeneous material, small strain analysis, static loading as typically assumed in linear elastic theory), but they by themselves cannot be used to predict performance directly (NCHRP1-37A, 2004).
The advantages of mechanistic-empirical methods are the improvement in the reliability of a design, the ability to predict the types of distress, and the feasibility to extrapolate from limited field and laboratory data.

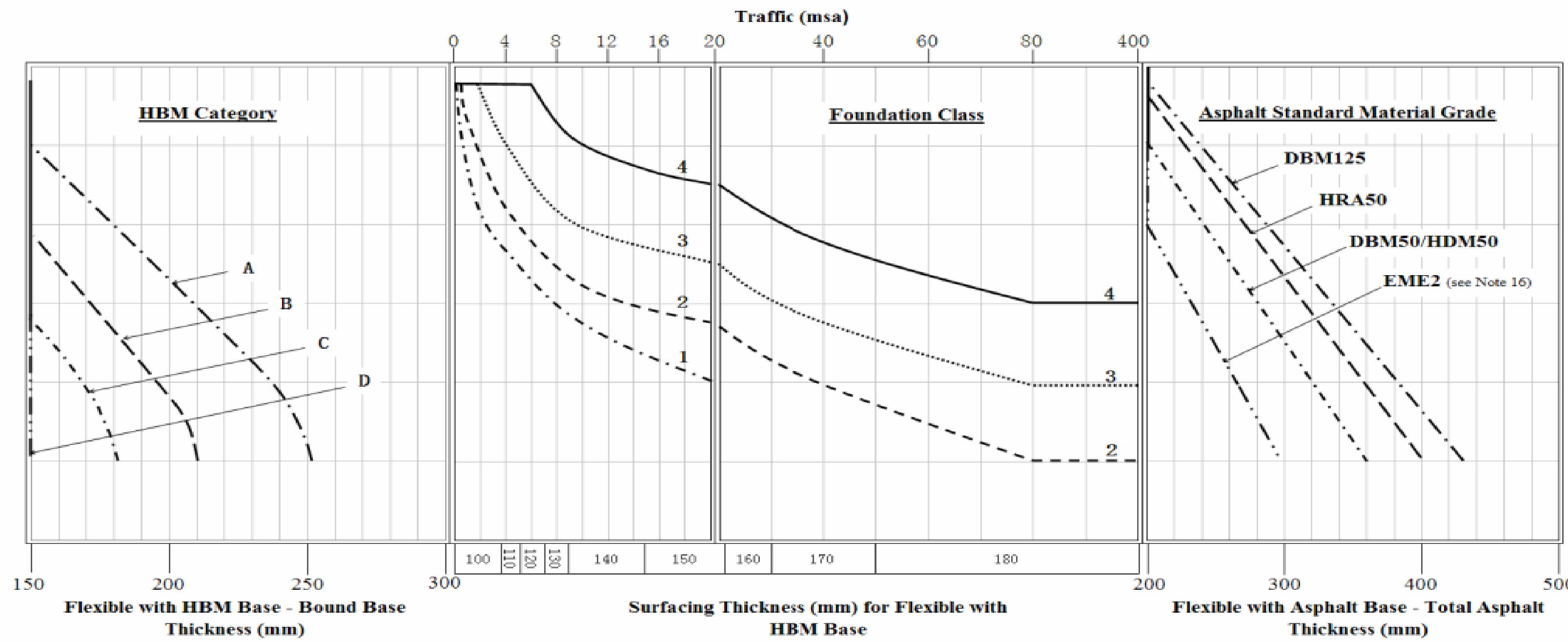
4.6 EXAMPLE OF EMPIRICAL METHODS

Pavement design and maintenance has developed from a combination of practical experience, laboratory research and full-scale road trials.
Nowadays, the pavement design standard in UK does not include a large range of design options (TRL 2004).
The predominant procedure currently in use for pavement design in the United Kingdom is the Design Manual for Road and Bridge 2006 Part 3 HD 26/06 guide.
The UK pavement design method described in HD 26 (Design Manual for
Roads and Bridges, Volume 7) is semi-empirical (analytical alternatives are allowed).

The design approach is based on four different foundation class defined in terms of the equivalent half-space stiffness based on the followed ranges:

- Class 1 ≥ 50 MPa: is a capping only design that has adequate shear strength. The use for high traffic roads may need to be limited.
- Class 2 ≥ 100 MPa: only sub-base or sub-base on capping design
- Class 3 ≥ 200 MPa: foundation of superior quality that provide the use of HBM. Could permit thinner overlying pavements.
- Class 4 ≥ 400 MPa: like the previous

From the nomographs, shown in Figure 55 it is possible to obtain the design thickness of the layers. For different classes of resistance four curves are plotted. The type curve A is referred to HBM which have the minimum class of resistance. In particular, for a CBGM B the minimum class is C8/10 as just discussed in the first part of this work.



Examples of Hydraulic Bound Base Materials

HBM Category	A	B	C	D
Crushed Rock Coarse Aggregate: (with coefficient of thermal expansion <10 × 10 ⁻⁶ per °C)	-	CBGM B – C8/10 (or T3) SBM B1 – C9/12 (or T3) FABM1 – C9/12 (or T3)	CBGM B – C12/15 (or T4) SBM B1 – C12/16 (or T4) FABM1 – C12/16 (or T4)	CBGM B – C16/20 (or T5) SBM B1 – C15/20 (or T5) FABM1 – C15/20 (or T5)
Gravel Coarse Aggregate: (with coefficient of thermal expansion ≥10 × 10 ⁻⁶ per °C)	CBGM B – C8/10 (or T3) SBM B1 – C9/12 (or T3) FABM1 – C9/12 (or T3)	CBGM B – C12/15 (or T4) SBM B1 – C12/16 (or T4) FABM1 – C12/16 (or T4)	CBGM B – C16/20 (or T5) SBM B1 – C15/20 (or T5) FABM1 – C15/20 (or T5)	-

Figure 55 Design thickness for flexible pavements

Most of the research has been carried out over a number of years by the Transport Research Laboratory (TRL), some with the assistance of external research contracts (H24/06, 2006).
For this reason HD 26/06 is based on TRL Report 615 (2004) for flexible and flexible composite construction which suggests the adoption of the material specific calibration adjustment factors that are close agreement with the previous flexible and flexible composite design, which were based on TRL report LR1132 (1984).

In "*Development of a more versatile approach to flexible and flexible composite pavement design*" (TRL615, 2004) Nunn offers a solution to improve the versatility of the current methods (LR1132, 1984).

This method predicts a compatible pavement layer thickness with existing methodology developing design criteria for hydraulically bound and bitumen-bound materials (TRL, 2004).
The standard conditions used for this versatile approach are the followed:

1. Linear elastic model response
2. Full bound for all
3. layers
4. Stresses and strains are induced under a single standard wheel load of 40 kN represented by a circular patch of 0.151 m radius with an uniform vertical stress
5. The calculations are based on equivalent pavement temperature of 20° C
6. 360 days values for dynamic modulus and flexural strength. This adoption bring the characterisation of HBM more in line with asphalt material enabling slow allowing both slow curing.
7. The structural properties of layers are shown in Table 19:

Table 19 Properties of layers

Layer	Poisson's Ratio
Asphalt	0.35
Hydraulically Bound Material	0.20
Foundation	0.35

For simplicity will be considered the traffic-induced stress σ_r at the underside of the hydraulic base determined using linear elastic theory. In order to take into consideration the temperature affects, curing behaviour and transverse cracking characteristic a k_{Hyd} calibration factor is used.

As shows Equation 6 it is determined empirically by sufficient performance data from in-service pavement. The report TRL615 suggests the adjustment factors for different classes and types of HBM relating the dynamic elastic modulus E (in GPa) and flexural strength f_f (in MPa).

$$k_{Hyd} = 0.368 + 5.27 * 10^{-4}E - 0.0351f_f$$

Equation 6 Adjustment factors k_{Hyd}

Exist a relationship between the static stiffness E_c and dynamic stiffness developed by Paul Edward (Edward , 2007) shown in Equation 7:

$$E_d=0.6E_c$$

Equation 7 Dynamic stiffness

Another factor used in this method is the K_{Safety}. This is used to control the inherent risk in pavement design. The default value is 1.0.
It is assumed a relationship (see Equation 8) between the dynamic modulus in GPa and the flexural strength in MPa developed in 1997 by Croney:

$$E = \frac{Log(f_f) + a}{b}$$

$$f_f = c * f_c$$

Equation 8 Dynamic modulus and flexural strength (360 days)

Where f_c is the compressive strength in MPa and *a, b* and *c* are material constants. These constants are different for gravel and crushed rock aggregates. The values are shown in Table 20:

Table 20 Values of constants

Aggregates	a	b	c
Gravel (G)	0.773	0.0301	0.11
Crushed Rock (R)	0.636	0.0295	0.16

As just said, this approach adopts 360 day as design value.
For the compressive strength of CBM, the method offers a relationship to estimate 360 day values from 28 or 7 day values, see Table 21:

Table 21 Conversion factors

Curing period	Relative compressive strength of CBM
7 day	0.67
28 day	0.80
360 day	1.00

The HD26/06 gives the stiffness modulus of standard UK asphalt material for use in analytical design:

Asphalt material	Stiffness modulus [MPa]
DBM125	2,500
HRA50	3,100
DBM50/HDM50	4,700
EME2	8,000

Using all of these parameters it is possible to calculate the million standard axles (msa) which the design is expected to support. The Equation 9 is shown below:

$$Log(N) = 1.23 * \left(\frac{f_f}{\sigma_r} * K_{Hyd} * K_{safety} + 0.1626\right)^2 + 0.2675$$

Equation 9 Life of pavement in million standard axles

With this value, the minimum thickness of asphalt cover is determined to reduce the risk of reflection cracking to an acceptable level through the following empirical Equation 10:

$$H_{Asphalt} = -16.05 * \left(Log(N)\right)^2 + 101 * Log(N) + 45.8$$

Equation 10 Thickness of asphalt

MECHANISTIC EMPIRICAL METHODS

This method is really useful during the project level because gives a powerful tool for design of the pavement structure.

Various mathematical models can be used to find a relationship between these phenomena and their physical causes. The most common theoretical basis adopted is a multi-layer elastic although pavement materials do not have a simple behaviour assumed in isotropic linear-elastic theory. Finite element and viscoelastic layer theory have limited use, possibly because of the difficulty in obtaining the required materials input and the complexity involved (Dalla Valle,

2015). Stress, strain and deflection within a pavement structure are the key words for pavement design and the loads and the material properties of the pavement layer are the physical causes.
To obtain the layer characteristic in terms of stress, strain and deflection exist a lot of computer based packages for mechanistic analysis. A summary listing of some of the more well know programs is shown in Table 22.

Table 22 Programs for Pavement Design (Montusci, 2011)

Program	Theoretical basis	Program source
CHEV5L	MLE	Chevron Research
BISAR	MLE	Shell International
ELSYM MLE	MLE	FHWA
PDMAP (PSAD)	MLE	NCHRP Project 1-10
JULEA	MLE	USACE WES
CIRCLY	MLE	MINCAD, Australia
VESYS	MLE o MLVE	FHWA
VEROAD	MLVE	Delf Technical University
ILLIPAVE	FE	University of Illinois
FENLAB	FE	University of Nottingham
SAPSI-M	Layered, damped elastic medium	Michigan State University/University of California Berkeley

Key: MLE – multilayer elastic, MLVE – multilayer viscoelastic, FE – finite element

Details and a summary of the most important features on the use of the BISAR 3.0 are presented in the next section.

4.7 BISAR 3.0

In the early 1970s, Shell Research developed the BISAR (Bitumen Stress analysis in Roads) mainframe computer program, which used in drawing the design charts of the Shell Pavement Design Manual issued in 1978.

Nowadays BISAR is widely used in the UK for analytical design procedures. BISAR is one amongst many available multilayer programs capable of calculate stresses and strains in the pavement structure. This program is a specific software package used to calculate deflections and is able to deal with horizontal forces and slip between the pavement layers. This offers the opportunity to calculate comprehensive stress and strain profiles throughout the structure for a variety of loading patterns (Pearson , 2011).

The limitations of this program are that all input parameters need to be inserted manually by user and also that it is possible to analyse maximum 10 structures at the same time.

With the BISAR program, stresses, strains and displacements can be calculated in elastic multi-layer system which is defined by the following configuration and material behaviour (Shell, 2000):

1. The system consist of horizontal layers of uniform thickness resting on a semi-infinite base of half space;
2. The layers extend infinitely in horizontal directions;
3. The material of each layer is homogeneous and isotropic;
4. The materials are elastic and have a linear stress-strain relationship.

Despite the simplifications used by multilayer linear elastic theory, this software can be achieved by using linear elasticity in order to predict pavement structural response as show a number of studies of Hildebrand (2002), Mateos et al. (2008), Mateos & Snyder (2002) as cited in Mateos et al (2013).

The data required are listed below.

LOAD: data for the load on the pavement and the radius of the loaded wheel are required in order to calculate the area of contact, the force applied and the stresses and strains in selected positions (see Figure 56). It is possible to select various types of wheel options like standard dual wheel, standard single wheel and super wheel.
There is also the option to specify is horizontal and/or shear forces are applied (Shell, 2000).

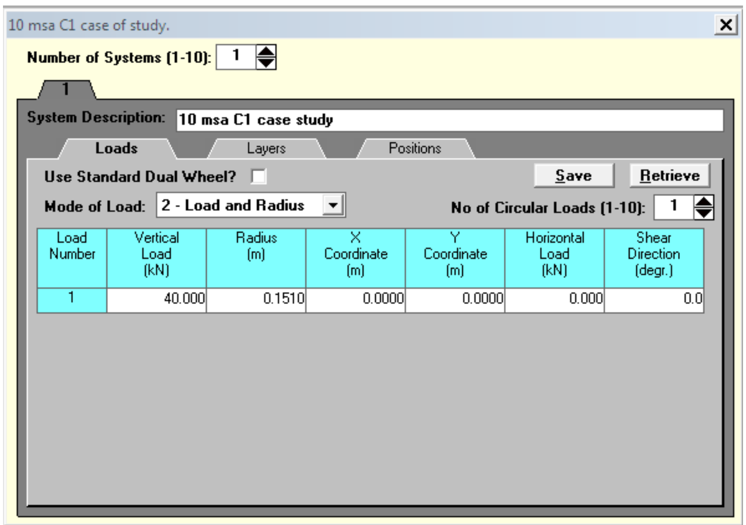


Figure 56 Parameters required

LAYERS: as shows Figure 57 the programme required data for the thickness of the layers, Poisson's ratio and number of the layers.
Therefore there are an options to specify if exist the full friction between layers and also if slippage between layers exist.

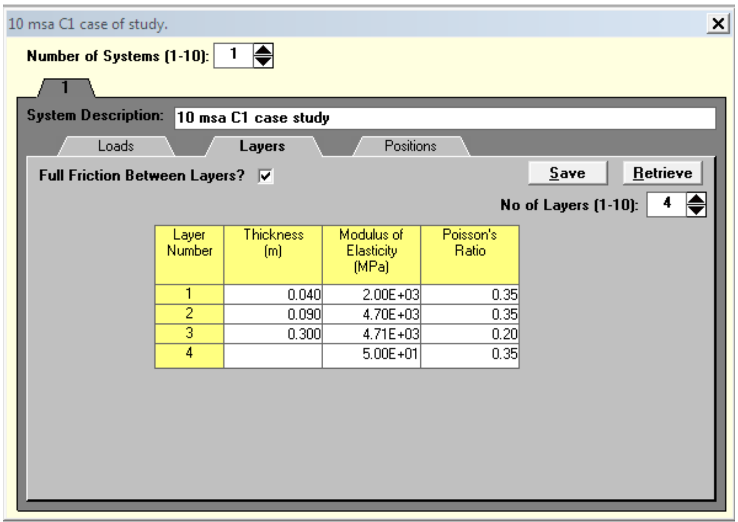


Figure 57 Layers' requirements

POSITION: the position where the stresses and strains are calculated can be selected as a standard option or the x, y and z coordinate positions can be specified. The inputs required are displayed in Figure 58.
To facilitate SPDM related calculations the present BISAR package contains options to access with ease the standard wheel configuration and automatically select important positions in the layer structure under construction (Shell, 2000).

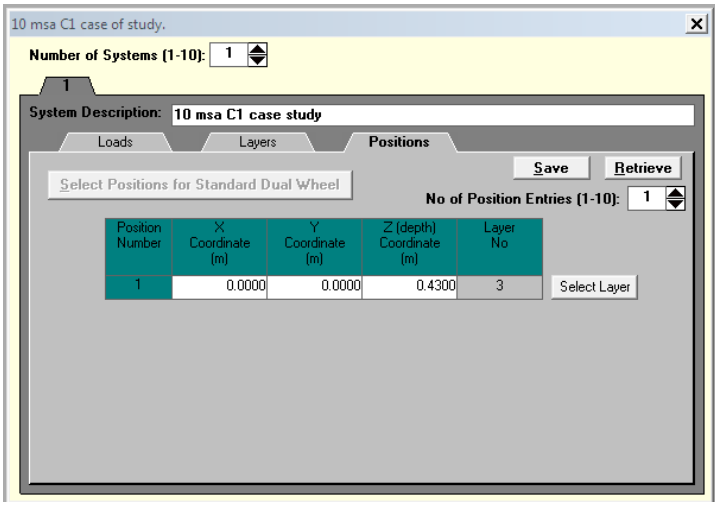


Figure 58 Positions required

The system is loaded on top of the structure by one or more circular loads, with a uniform stress distribution over the loaded area. The effect of vertical stresses (shear forces at the surface) are calculated by this programme and also it includes an option to account for the effect of (partial) slip between the layers, via a shear spring compliance at the interface.

Summary BISAR calculations require the following input:

- The number of layers
- The Poisson's rations of the layers
- The thickness of the layers (except for the semi-infinite base layer)
- The interface shear spring compliance at each interface
- The number of loads
- The co-ordinates of the position of the centre of the loads
- One of the following combinations to indicate the vertical normal component of the load
- Stress and load
- Load and radius
- Stress and radius
- The co-ordinates of the positions for which output is required.

5. DESIGN METHODOLOGY

The tested Cement bound granular material gives different values of compressive strength and stiffness. The first is very slow compared to the high stiffness.
From the laboratory tests the respective values were found:

$$R_{c\text{-}28}=3.3 \text{ MPa} \qquad E_c=23.543 \text{ GPa}$$

The pavement design study was conducted following two different steps in order to ensure the correct methodology suggested by HD26/06:

- Step 1

The $R_{c\text{-}28}$ was used as start value. Following the requirement of TRL615 this value must to be referred on 360 days. Using the conversion factor of 0.8 shown in Table 21 $Rc_{\text{-}360}$ was calculated.
Finally, from Equation 8 the values of flexural strength f_f and E were found. For the E values the constant values relative on gravel material were used (see Table 20):

$$a=0.773 \qquad b=0.0301 \qquad c=0.11$$

The results are summarized in Table 23:

Table 23 Results step 1

Parameter	Dimension	Result
$R_{c\text{-}28}$	MPa	3.3
$Rc_{\text{-}360}$	MPa	4.1
f_f	MPa	0.45
E	GPa	14.28

- Step 2

The E_c values was fixed as start value. Following the requirement in TRL615 this value must be referred to 360 days in terms of dynamic modulus. Using the Equation 7 (Edward , 2007) and the conversion value of 0.9 (TRL615, 2004) the dynamic modulus E was calculated. Using the Equation 8 the flexural strength was determined. The results are shown in Table 24:

Table 24 Input step 2

Parameter	Dimension	Result
E	GPa	15.695
f_f	MPa	0.5

From the results it is evident that the E values do not differ significantly from each other despite two different followed steps. This means that the inputs selected were appropriate to characterise this material.

From the nomograph in Figure 55, corresponding with the curve A (CBGM B C8/10), the thicknesses of the asphalt and base layers were found for different values of traffic and foundation class.
Four different traffic values were analysed:

- 10 msa
- 20 msa
- 40 msa
- 80 msa

The achieved thicknesses were used to determine the start thicknesses values for the material which was being investigated.
Four pavement class foundations were used in a multilayer linear elastic analysis, to study the response of each structure to the application of a standard wheel load.
The model of pavement structure used in the design was a semi-infinite multi-layer elastic system presented in Figure 59:

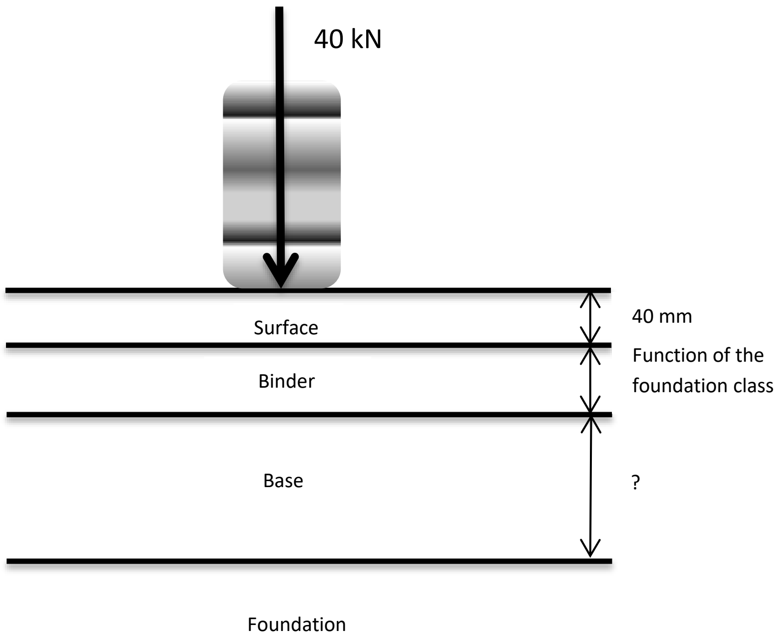


Figure 59 Typical pavement structure used in BISAR

As just discussed in the literature review, the software Bisar 3.0 needs an input parameters. They were obtained from TRL Report 615 and HD26/06. Table 25 shows the adopted inputs for each step.

Table 25 Input parameters for Bisar 3.0

Input	Layer	Step 1	Step 2
Load		40 KN	
Radius		0.151 m	
Thickness	Thin surfacing	40 mm	
	Binder	By graph Figure 55	
	Base	Unknown	
	Foundation	Infinite	
Stiffness	Thin surfacing (DBM50)	2000 MPa	
	Binder (DBM50)	4700 MPa	
	Base (CBGM)	14280 MPa	15189 MPa
	Foundation	C1=50 MPa C2=100 MPa C3=200 MPa C4=400 MPa	
Poisson's ratio	Thin surfacing	0.35	
	Binder	0.35	
	Base	0.2	
	Foundation	0.35	
Position		On the bottom of the base layer	

Several iterations were made for each base course, consecutively increasing or decreasing the thickness of its layer H_{CBGM}, and determining the fatigue life, according to the stress obtained. This procedure was carried out until reaching the expected fatigue life (msa) according to the input parameters.

The first step was to choose the inputs needed to carry out this analysis.
Four different traffic classes were analysed with Bisar 3.0 with different foundation stiffness and surface thicknesses. For each step, a total of 16 case scenarios were conducted.
As just discussed in the previous paragraph, following two different ways, the final results were similar in terms of stiffness concluding that the inputs were appropriate for this investigation.
Although it worked, it is important to point out that the outputs are the results of some assumptions:

- The total thickness of asphalt was obtained from the right hand portion on the nomograph (Figure 55) which is based on specific types of materials not directly comparable with the material which is subject study;
- The constants values and the suggested ratio between the 28 and 360 days for standard CBM grades shown in Table 20 were considered valid for this specific analysis.

The flow chart in Figure 60 resumes the followed procedure:

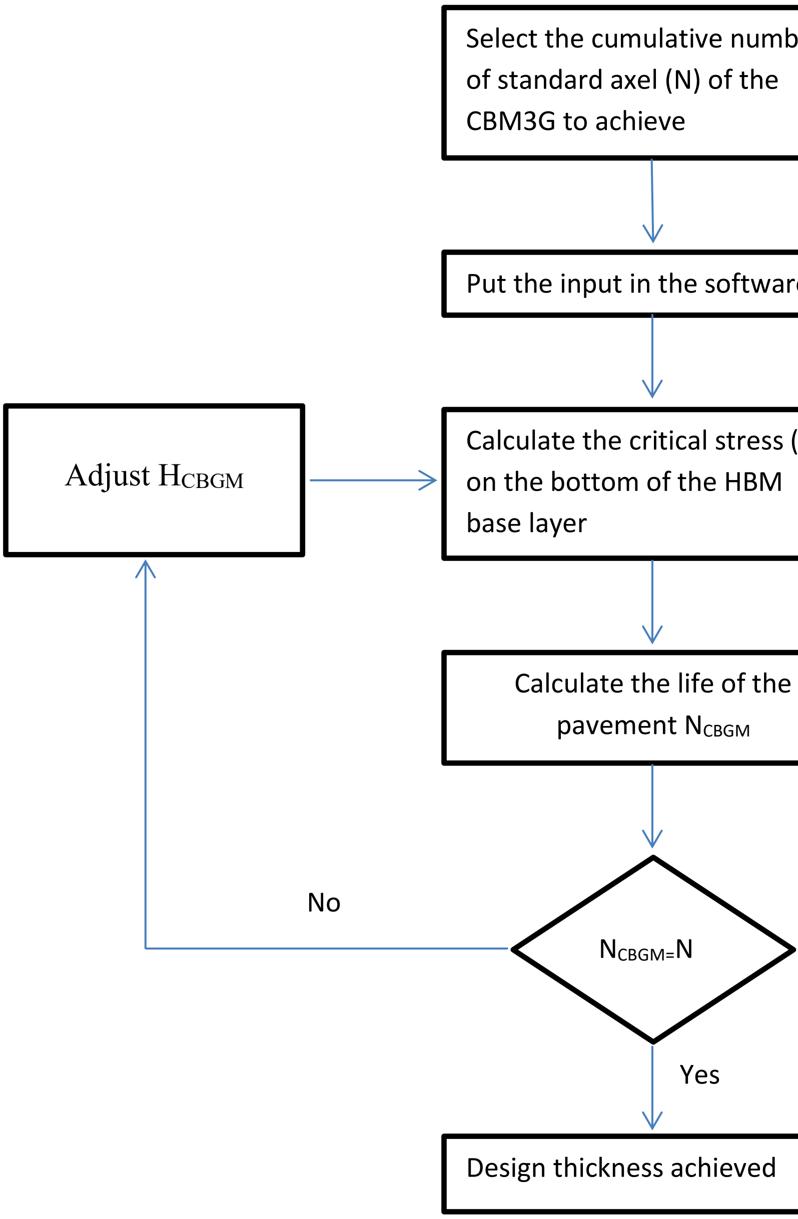


Figure 60 Flow chart of the methodology

6. RESULTS

The analytical pavement design thickness results using Bisar 3.0 and TRL615 and HD26/06 stiffness/flexural strength models are plotted in Table 26 and Table 27:

Table 26 Results following step 1

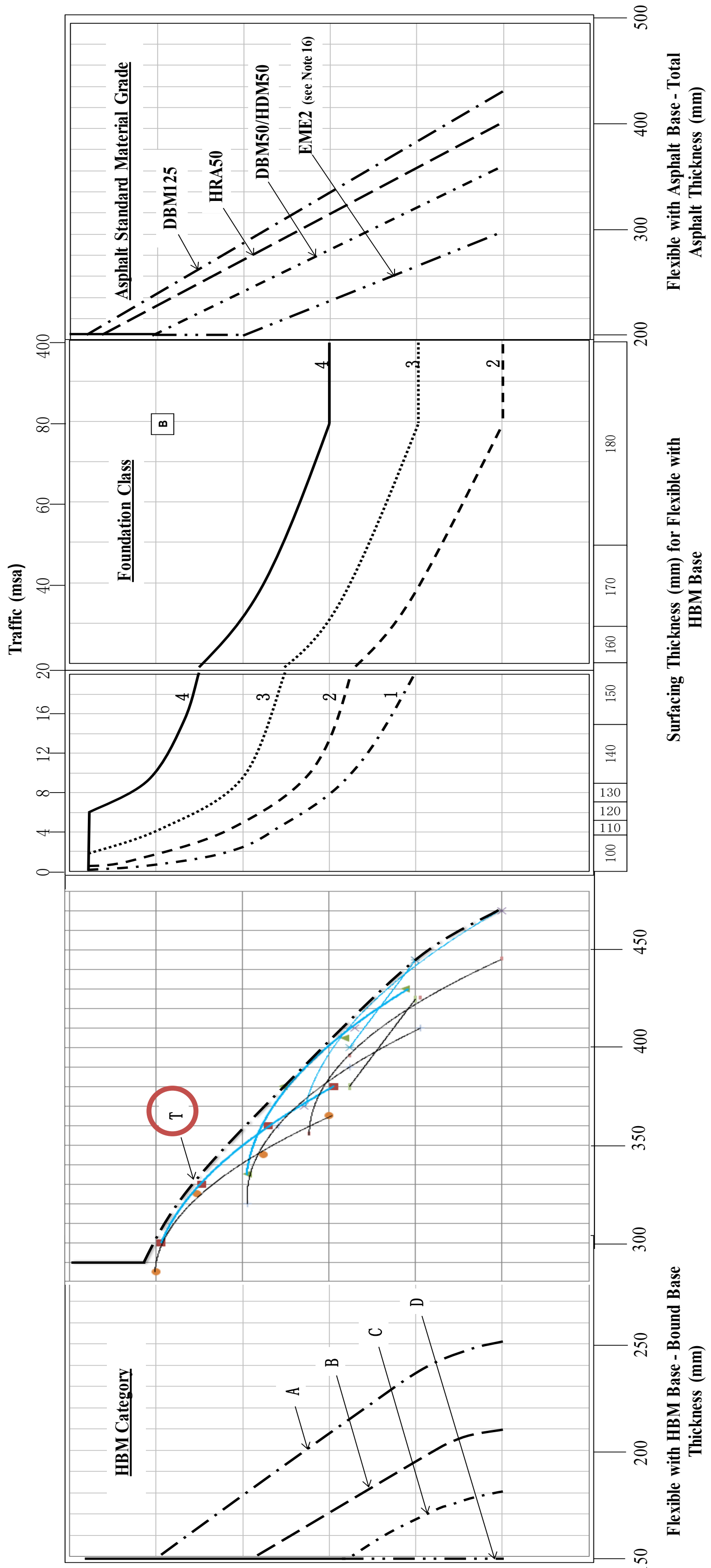
Material	Traffic (msa)	Foundation Class	Stiffness Foundation (MPa)	Thickness Base (mm)	Thickness surface (mm)	Stress (MPa)	Life (msa)
CBGM B C2,3/3	10	C1	50	400	130	0.265	10.3
		C2	100	370		0.266	10.2
		C3	200	335		0.267	10.1
		C4	400	300		0.263	10.5
	20	C1	50	445	150	0.213	21.3
		C2	100	410		0.215	20.5
		C3	200	380		0.210	22.6
		C4	400	330		0.216	20.1
	40	C1	50	480	165	0.183	43.1
		C2	100	445		0.183	43.1
		C3	200	405		0.183	43.1
		C4	400	360		0.184	41.9
	80	C1	50	505	180	0.163	85.7
		C2	100	470		0.162	89.3
		C3	200	430		0.162	89.3
		C4	400	380		0.163	85.7

Table 27 Results following step 2

Material	Traffic (msa)	Foundation Class	Stiffness Foundation (MPa)	Thickness Base (mm)	Thickness surface (mm)	Stress (MPa)	Life (msa)
CBGM B C2/3	10	C1	50	380	130	0.294	10.8
		C2	100	355		0.291	10.4
		C3	200	320		0.295	10.5
		C4	400	285		0.292	10
	20	C1	50	425	150	0.234	21.2
		C2	100	395		0.234	21.3
		C3	200	360		0.234	21
		C4	400	325		0.229	20.6

Material	Traffic (msa)	Foundation Class	Stiffness Foundation (MPa)	Thickness Base (mm)	Thickness surface (mm)	Stress (MPa)	Life (msa)
	40	C1	50	455	165	0.200	40.7
		C2	100	425		0.203	43.8
		C3	200	390		0.202	43.5
		C4	400	345		0.200	44.1
	80	C1	50	480	180	0.181	80.7
		C2	100	445		0.181	80.7
		C3	200	410		0.179	86.8
		C4	400	365		0.179	86.8

For each step the results were plotted on the right hand portion of the normograph. From Figure 61 it is possible to view the final drown curve for a Cement Bound Granular Material with a resistance class of 2,3/3MPa.



HBM CATEGORY	T	A	B	C	D
Gravel Coarse Aggregate	CBGM B - C2,3/3	CBGM B - C8/10	CBGM B - C12/15	CBGM B - C16/20	-

Figure 61 New Version of the normograph

7. CONCLUSIONS

Within my research, a laboratory characterization has been carried out on site-won sand and gravel material. The aim was to ascertain the possibility to use this material as hydraulically bound material (HBM) in the base layer of a new pavement construction and analysing the pavement design in accordance with these conditions.

PSD results showed some variation between sample locations but the requirements of a CBGM material could be also met by removing aggregates above 40mm. Therefore, the addition of cement boosted the lower part of the curves, making them fall inside the required envelope.

The moisture content of the samples was variable due to the different positions. To avoid variability in the test results due to this moisture content, aggregates were dried before manufacturing the mixtures.

The optimum moisture content was determined as 4.5%; therefore, the mix design was based on production of specimens at 4.5 and 6.5% water. Cement contents were selected at 4%, 5% and 6% based on the previous experiences of CBGM material by using gravel aggregate.

In terms of aggregates quality, all requirements were met for manufacturing a CBGM, exception made for the plasticity index. If the material is variable and the clay is localised, it may be eliminated and the rest of the materials can be used as a CBGM. However, if this is not possible, the site-won gravel might not be used as a CBGM in the base course of a pavement.

The mix design results showed a low strength material (C2,3/3) which may not be suitable for being used as CBGM in the pavement base as HD26/06 requires. In compliance with the pavement layer thickness, this guide suggests the use of a CBGM having strength classes of at least C8/10 or C9/12.

To understand why this aggregate it is not suitable for a base pavement layer, two different approach were used in order to analyse the pavement design, due to the variability between the strength and the stiffness.
For each step, the results were plotted on the right hand portion of the normograph. From Figure 61 it is possible to see the final drawn curve for a Cement Bound Granular Material with a class of resistance of C2,3/3.

A conservative approach was used for drawing the final curve. All the points were situated below of it, in order to satisfy the two different analysed steps.

The relative CBGM C2,3/3 thickness should be increased 93% compared to a CBGM 8/10. This demonstrates that these aggregates are not suitable for being used in a base layer. For this reason, strengths obtained were not sufficient to produce a CBGM, and, the higher cement contents required to increase the strength may be uneconomic.

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APPENDIX A