University of Studies of Cagliari

PhD

in STRUCTURAL ENGINEERING

Cycle XXVIII

Use of Local Available Material for Inverted Pavement Technique

Learning scientific field of reference

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ABSTRACT

This research has the purpose to build infrastructures by using new techniques, new materials and to start a sustainable design with the utilization of local resources. The project also aims to minimize the use of non-renewable sources as hot mix asphalt.

Concerning the materials, it is particularly highlighted the use of stone materials available in the Region of Sardinia, currently extracted to create infrastructures and bitumen layers. New techniques have been studied to build up infrastructures such as the Inverted Pavement Technique used in South Africa for more than 30 years. In particular we focused on the realization of the G1 Layer for this paving.

Through this work, we are willing to determine the possible use of the materials present in Sardinia to build Inverted Pavement infrastructure. The target is to better use the local obtainable resources in terms of aggregates, trying to make the most of their mechanical and physical features.

During the research, a great attention was focused on the illustration of different typologies of paving currently used in Europe and in South Africa, specially those realized through base layers with granular material, examining in depth the anisotropic property of granular bases too. A panoramic of the property of road construction material used in South Africa is presented, by analyzing the issues linked to the type of minerals that are in the rocks and the way through which they affect the resistance and durability of one layer of the paving.

In the first phase of the study, we concentrated on the specification of the kinds of materials mined in Sardinia and their location. Then, we chose to investigate about the following materials: andesite, basalt, quartzite, trachite, dolomite and ignimbrite. All these are materials with good features such as mechanical endurance and durability.

An in-depth bibliographic research was done about the 'Inverted Pavement' technique for understanding the behavior of this type of paving during the useful life, compared to a traditional paving. The building technique of the G1 layer was mainly examined, because some phases, such as that called "slushing", are considered complex and the main point for the good result of layer and the whole superstructure.

To make sure the material is suitable, we referred to South-African regulations and we carried all the mechanical tests foreseen to categorize it as G1. Chemical analyses and thin sections were analyzed, because they are considered preliminary characterizations on the study of materials. Some triaxial tests were performed with several confining pressures to define the elastic modulus and other strength parameters.

Anyway it is not enough to comparison chemistry and petrography to establish that a material is suitable. For this reason, were done other physical-mechanical laboratory tests. Two test sections of Inverted Pavements realized in Atlanta, using granite, were examined and were done some laboratory tests on material. From the monitoring of these two test sections and from the result of tests, it emerged that Inverted pavement has performances greater than conventional paving and that other kinds of materials can be used for realization.

1. INTRODUCTION

Aggregates used in road design are a mix of grains which has growing mechanical performance depending on the energy and compaction pressures. The need of creating stable and durable civil engineering works is closely related to the ability of compacting aggregates as more stringent possible. The development of compaction instruments (energy applied to the aggregates) has opened new and large perspectives that nowadays are still under study. Already since the 80's in South Africa technical regulations for the construction of infrastructures have foreseen the use of materials with high resistance to loads of static and dynamic type that allow to build paving with low economic costs and high environmental value, limiting the use of hot mix asphalt only to the top layer with restrained thickness.

The current economic situation has severely affected the Italian road infrastructure, and this affects both maintenance and future growth. The inverted base pavement structure is a promising alternative to achieve high quality roads at considerably lower cost than conventional pavements.

The Italian transportation system is a vast network of roads, railways and airports. The road network is 25.528,781 kilometres of existing paved roads and 80% of this are managed by ANAS. Growth rate of approximately 56,000 lane-kilometres.

Currently, the "Decree of Doing" (Decreto del Fare) foresees paving maintenance interventions for a total amount of Euro 14.920.257,00 of which 4.238.267,00 just for Sardinia.

In the ANAS program contract are expected new road works for about 650 M € and new projects for 7,75 M €.

As shown above, the roads network requires high amounts of money. In this context, new alternatives with lower life-cycle costs would be welcome.

1.1. Aims and Objectives

The aims of the research can be summarized as it follows:

- Determine which materials extracted in Sardinia could be used for the realization of the G1 layer of the IP (Inverted Pavement)
- Analyze the construction process of the G1 layer
- Make sure the selected materials meet the minimum requirements of the South African regulation for the G1 layer.

Specific objectives of the research:

- Describe which are the paving kinds currently used in Europe and in South Africa
- Describe how to obtain high densities of the G1 layer.
- Analyze what is known about ‘Inverted Pavement’ and what is the experience reached using materials different to those used in South Africa
- Evaluate, according to laboratory tests, how material is inserted into the South African Regulations

1.2. Thesis lay out

This thesis is divided in the following chapters:

Chapter 1: short description of the context where the research is inserted

Chapter 2: short description of different pavement designs like flexible pavement design, rigid pavement design in European Country.

Chapter 3: short description of different pavement designs like flexible pavement design, rigid pavement design in South Africa.

Chapter 4: provides an extensive compilation of documented inverted base pavements in the US.

Chapter 5: reports of stress-dependent response of coarse granular materials.

Chapter 6: provides an extensive description of properties required for road construction material in terms of geometry, weathering and mineral behavior.

Chapter 7: selection procedures for granular material available in the region.

Chapter 8: reports a comprehensive laboratory investigation designed to study chemical-physical behavior of selected rocks.

Chapter 9: reports a triaxial test response of coarse granular materials.

Chapter 10: reports a comprehensive laboratory investigation designed to study mechanical behavior of selected rocks.

Chapter 11: reports about my experience at Georgia Institute of Technology and characterization material of Moran County and LaGrange test section.

Chapter 12: Conclusion.

Chapter 13: Recommendation for further research.

2. EUROPEAN ROAD PAVEMENTS DESIGN

2.1. Introduction

The pavement structure is a sequence of layer placed over the soil and directly subject to the action of the vehicles. It has three basic functions: 1) ensuring a smooth rolling surface and not many deformable; 2) distributing the loads transmitted by the vehicles on the sub-grade in order to avoid excessive deformation of the surface layer; 3) protecting the below layer from weathering (Ferrari & Giannini).

Looking at the functional aspect of the pavements structure, it is constituted by a surface layer and a supporting structure. The surface layer is directly subject to the actions of the traffic and the weathering. The supporting layers has the only function of keeping unaltered the geometrical configuration of the surface layer and of distributing to the sub-grade the stresses caused by traffic (Garbin, Storoni, & Ridolfi).

Road pavements are typically formed of several layers, one above the other, consist of mixtures of stone aggregates, binders and concrete. The most used binders are bitumen and cement, the first one is used in the flexible pavements whilst the second one is used in the semi-rigid and rigid pavements.

A traditional classification recognized the road pavements as flexible, rigid and semi-rigid; the difference among them is about how they distribute the traffic load to the sub-grade.

2.2. Traffic Category

Road is an area with a public employment, used for the pedestrians, vehicles and animals movement. (Infrastrutture, 2001). The functional qualification of roads is based on the kinds of users and activities allowed on the roads themselves, considering the environmental situation where they are inserted. The criteria of planning concern the geometric elements of the axle and of the platform of the urban and extra-urban roads, such that the movement of users, who are allowed, happens through security and regularity. Specifically, for the motorized vehicles, these rules pursue the purpose of persuading drivers not to go beyond the values of speed that are placed at the base of the planning.

The transport request, identified by the time volume of traffic, by its composition and by the average speed of flow, implicates, as a planning choice, the road section and the interval of project speed. In particular, the choice of the number of lanes of the road section and of their typology determines the traffic offer, whereas the choice of the interval of project speed influences, based on the space crossed by the infrastructure, the plano-altimeter features of the axle and the sizes of the various elements of section. The expression "interval of project speed" means the range of values based on which the features of the various elements of road layout have to be specified (straights, circular curves and curves with a variable radius). These values change from an element to another, with the purpose of allowing the designer to have a specific freedom to adapt the layout to the area that is crossed.

Some basic factors were identified and those, characterizing the road networks from the functional point of view, allow to situate the network, that is the subject matter of the research, in a precise class; they are:

• the kind of movement supplied (with passage, distribution, penetration and entry); the movement is considered in the opposite direction too, namely that of progressive collection at the various levels;

• the extent of the transfer (the distance which is gone through by vehicles on average);

• the function that is taken in the territorial context that is crossed (national, inter-regional, provincial, local connection);

• the traffic components and the respective categories (light vehicles, heavy vehicles, motorvehicles, pedestrians, ect.).

In order to arrive at the identification of the road areas necessary to the various traffic components, for fulfilling the functions that are expected in full compliance with the criteria of security and regularity of movement, the traffic components, the classes of vehicles and the functions that are allowed, have been gathered in traffic categories, homogeneous for functional features and needs, shown below:

- Motorbike
- Vehicles,
- Bus,
- Trailer
- Semitrailer,
- Tractor trailer,
- Articulated truck
- Operating machinery

2.3. Road Category

The New Road Code classifies roads in six different typologies, each identified by a letter from A to F:

A) Extra-urban motorways

B) Main extra-urban roads (a road external to inhabited centers)

C) Secondary extra-urban roads

D) Urban expressways (a road into the inhabited center)

E) Roads in residential areas

F) Local extra-urban roads and local urban roads

For each category of road it is allowed only a specific kind of traffic category, as it is reported in the Table 1. Where symbols have the following meanings:

o Not allowed in platform

❖ In roadway

☐ external to the roadway (in platform)

● partially in roadway

1. It affects if there is a bike path

2. If the categories number 7 and number 11 must be allowed, the sizes of lanes and the layout of axle must be commensurated with the needs of the vehicles belonging to these categories.

3. When there is a slip road, namely when the platform of the main road and of the service road is unique, not allowing on the main road means that this is limited only to the part of platform that concerns it.

Table 1 Types of road - permitted traffic categories

TIPI SECONDO IL CODICE	A AUTOSTRADA	A AUTOSTRADA	A AUTOSTRADA	A AUTOSTRADA	B EXTRAURBANA PRINCIPALE	B EXTRAURBANA PRINCIPALE	C EXTRAURBANA SECONDARIA	D URBANA DI SCORRIMENTO	D URBANA DI SCORRIMENTO	E URBANA DI QUARTIERE	F LOCALE	F LOCALE
AMBITO TERRITORIALE	EXTRAURBANO	EXTRAURBANO	URBANO	URBANO	EXTRAURBANO	EXTRAURBANO	EXTRAURBANO	URBANO	URBANO	URBANO	EXTRAURBANO	URBANO
DENOMINAZIONE	STRADA PRINCIPALE	STRADA DI SERVIZIO (EVENTUALE)	STRADA PRINCIPALE	STRADA DI SERVIZIO (EVENTUALE)	STRADA PRINCIPALE	STRADA DI SERVIZIO (EVENTUALE)		STRADA PRINCIPALE	STRADA DI SERVIZIO (EVENTUALE)			
CATEGORIE DI TRAFFICO												
1 PEDONI	○	□	○	○	○	□	□	○	○	○	□	○
2 ANIMALI	○	□	○	□	○	□	□	○	✸	◆	◆	◆
3 VEICOLI A BRACCIA E A TRAZIONE ANIMALE	○	◆	○	◆	○	◆	◆	○	◆	◆	◆	◆
4 VELOCIPEDI	○	◆	○	◆	○	◆	◆ □ (1)	□	◆	◆ □ (1)	◆ □ (1)	◆
5 CICLOMOTORI	○	◆	○	◆	○	◆	◆	◆	◆	◆	◆	◆
6 AUTOVETTURE	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆
7 AUTOBUS	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆ (2)
8 AUTOCARRI	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆
9 AUTOTRENI AUTOARTICOLATI	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆	◆	○
10 MACCHINE OPERATRICI	○	◆	○	◆	○	◆	◆	◆	◆	◆	◆	◆
11 VEICOLI SU ROTAIA	○	○	○	□	○	○	○	○	□	□	○	□ ◆ (2)
12 SOSTA DI EMERGENZA	□	□	□	□	✸	✸	✸	✸	✸	◆ ✸	□	□
13 SOSTA	○	□	○	□	○	□	□	○	□	□	□	□
14 ACCESSI PRIVATI DIRETTI	no	si	no	si	no	si	si	no	si	si	si	si

For each category of road was analyzed the traffic composition using some spectra that are typical of commercial vehicles, namely an overall mass bigger than three tonnes. In the Table 2 are reported the kinds of vehicles that were considered and their loads per axle; whereas, in the Table 3 is represented their frequency expressed per cent, on the total amount of trade means. (CNR, 2001).

Table 2 Types of commercial vehicles, number of axles, distribution of axle loads.

Tipo di veicolo	N° Assi	Distribuzione dei carichi per asse in KN
1) autocarri leggeri	2	↓10 ↓20
2) " "	"	↓15 ↓30
3) autocarri medi e pesanti	"	↓40 ↓80
4) " " "	"	↓50 ↓110
5) autocarri pesanti	3	↓40 ↓80 ↓80
6) " "	"	↓60 ↓100↓100
7) autotreni e autoarticolati	4	↓40 ↓90 ↓80 ↓80
8) " "	"	↓60 ↓100 ↓100 ↓100
9) " "	5	↓40 ↓80↓80 ↓80 ↓80
10) " "	"	↓60 ↓90↓90 ↓100↓100
11) " "	"	↓40 ↓100 ↓80 ↓80 ↓80
12) " "	"	↓60 ↓110 ↓90 ↓90 ↓90
13) mezzi d'opera	"	↓50 ↓120 ↓130↓130↓130
14) autobus	2	↓40 ↓80
15) "	2	↓60 ↓100
16) "	2	↓50 ↓80

Table 3 Typical traffic spectra of commercial vehicles for each type of road.

	Tipo di veicolo															
Tipo di strada	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1) autostrade extraurbane	12.2	----	24.4	14.6	2.4	12.2	2.4	4.9	2.4	4.9	2.4	4.9	0.10	----	----	12.2
2) " urbane	18.2	18.2	16.5	----	----	----	----	----	----	----	----	----	1.6	18.2	27.3	----
3) strade extr. principali e secondarie a forte traffico	----	13.1	39.5	10.5	7.9	2.6	2.6	2.5	2.6	2.5	2.6	2.6	0.5	----	----	10.5
4) strade extraurb. second. ordin.	----	----	58.8	29.4	----	5.9	----	2.8	----	----	----	----	0.2	----	----	2.9
5) " extr. second.-turistiche	24.5	----	40.8	16.3	----	4.15	----	2	----	----	----	----	0.05	----	----	12.2
6) " urbane di scorrimento	18.2	18.2	16.5	----	----	----	----	----	----	----	----	----	1.6	18.2	27.3	----
7) " " di quartiere e locali	80	----	----	----	----	----	----	----	----	----	----	----	----	20	----	----
8) corsie preferenziali	----	----	---	----	----	----	----	---	----	----	---	----	----	47	53	----

Traffic is expressed in a total number of passage of commercial vehicles passing through on the most loaded lane. The traffic levels, that were foreseen, are reported in the Table 4.

Table 4 Traffic levels on the more loaded lane

Livello di traffico	Numero di veicoli commerciali
1°	400.000
2°	1.500.000
3°	4.000.000
4°	10.000.000
5°	25.000.000
6°	45.000.000

Based on the traffic category, on the road category and on the traffic levels that were foreseen in the project, it will be made a redrafting of the superstructure in terms of typology and thickness of each layer. The typologies of paving are divided in:

- flexible pavings;
- rigid pavings;
- semi-rigid pavings;
- composite pavings.

They are presented with detail in the following paragraphs.

2.4. Flexible pavements

They are generally made by a surface layer, a connection layer (binder), a base layer and a foundation layer, as you can see in the Figure 1.

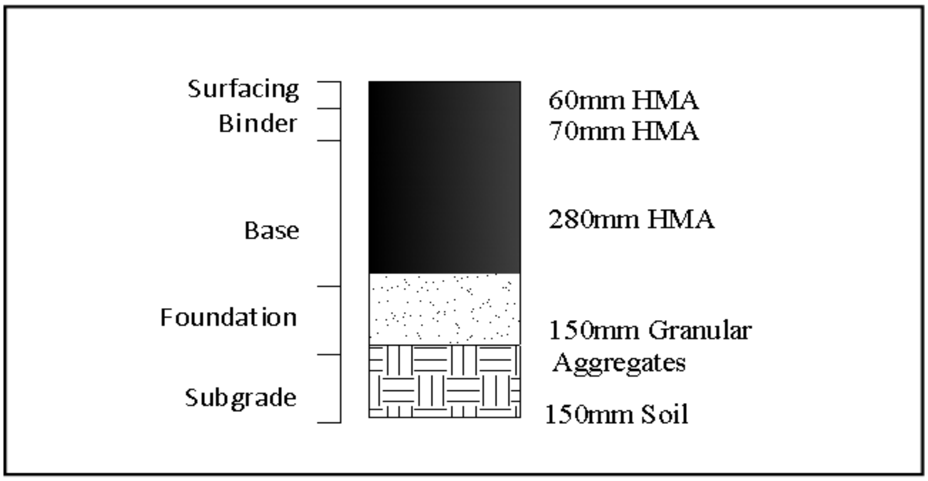


Figure 1 Italian Flexible Pavement

Surfacing:

It consists of a Hot Mix Asphalt, open or closed, with thickness ranging from 40 to 60 mm must ensure adherence, good resistance to bypass actions and vertical deformations.

The properties of a surface layer are to be related with the mineralogy and grain size of the aggregates and rheological properties of bitumen.

Binder:

It consists of Hot Mix Asphalt, semi-open, but with lower mechanical properties. It contributes, together with the base layer, to absorb the flexural action induced by traffic loads. The thickness used varies between 6 and 8 cm.

Base layer:

To the base layer is given the role of absorbing most of the flexural actions caused by traffic loads. A flexible pavement is made with hot mix asphalt, masses open, with less amounts of bitumen, higher porosity and mechanical characteristics lower than the previous layers. The thickness of a base layer is widely variable from a minimum of 8-10 cm to a maximum value of 25-30 cm, depending on the number of heavy traffic, weather conditions and the bearing capacity of the sub-grade.

Foundation layer:

Generally, it is composed by granular material and its main function is to distribute the loads on the sub-grade. The thickness range is very large: between 15 and 35 cm.

Sub-grade:

the success and the durability of the infrastructure depend on the behaviour of the sub-grade. The sub-grade is the soil bed on which it leans the foundation layer of the pavements, and it is not affected by the stress of the traffic load. The thickness varies between 50 and 100 cm.

The lift of the sub-grade is :

- good 1500 from N/cm2
- medium 900 from N/cm2
- low 300 from N/cm2

In case of values under 300 from N/cm2 the sub-grade must be reclaimed and replaced.

The Figure 2 refers to an extract of CNR Official Bulletin in which are present different types of flexible paving that could be done according to the resilient modulus of the background and to the intensity of the traffic.

The Figure 2 shows exclusively the 1F Paving Catalog relating to extra-urban highways type which is the one chosen to build Inverted Pavement infrastructure.

The parameter chosen for the sub-grade lift is the Resilient Modulus (Project Mr), assessable on a experimental tests basis by using AASHTO T274-82 standard.

This type of parameter was selected because it is the one that best represents the sub-grade behavior since it allows to keep in consideration the viscous reversible component of the deformation as well.

In case the Resilient Modulus could not be determined it may be used the available approximate correlations with CBR bearing ratio and the K reaction module (C.N.R, 1995)

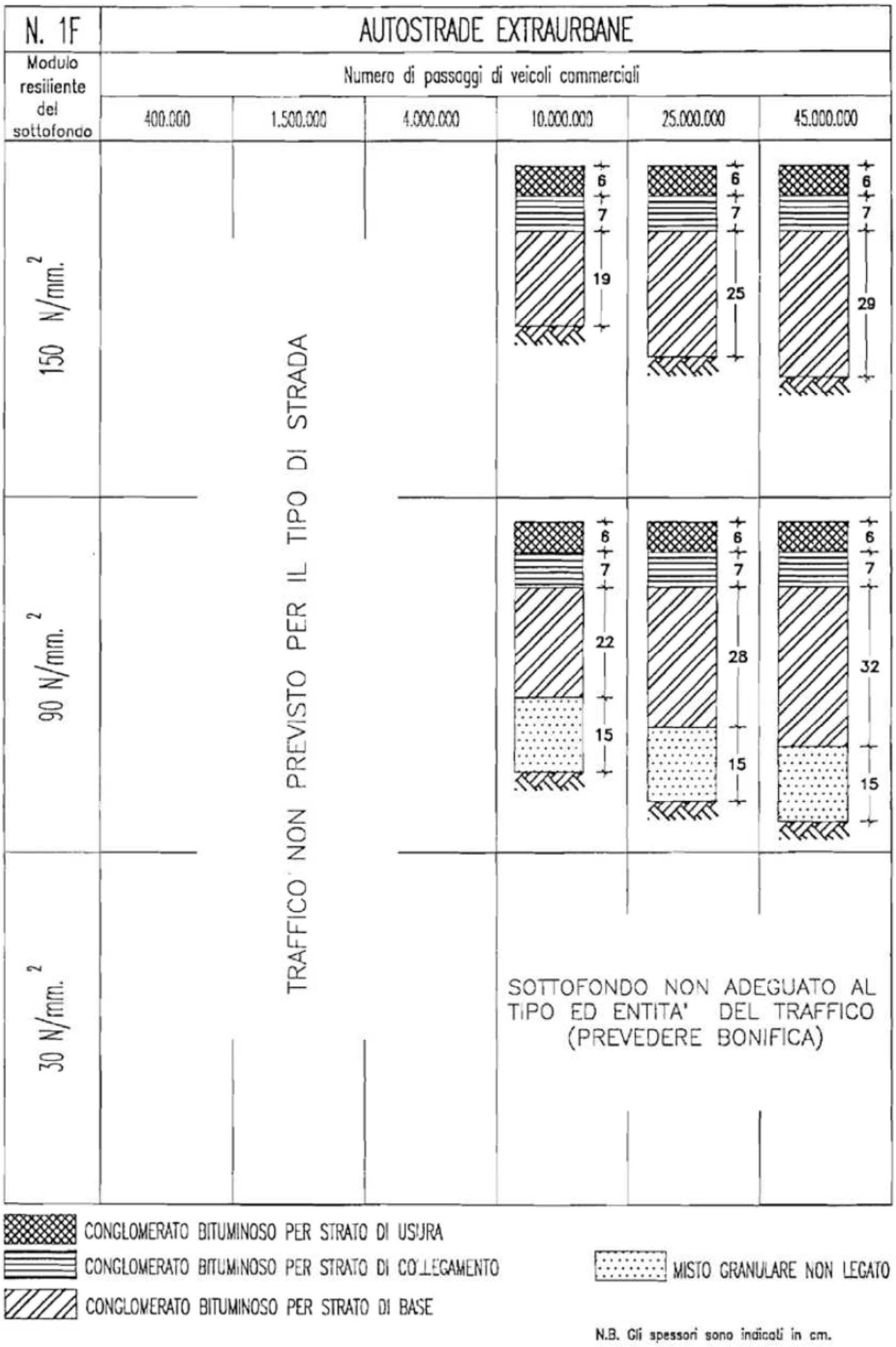


Figure 2 Italian Catalogue for Freeways

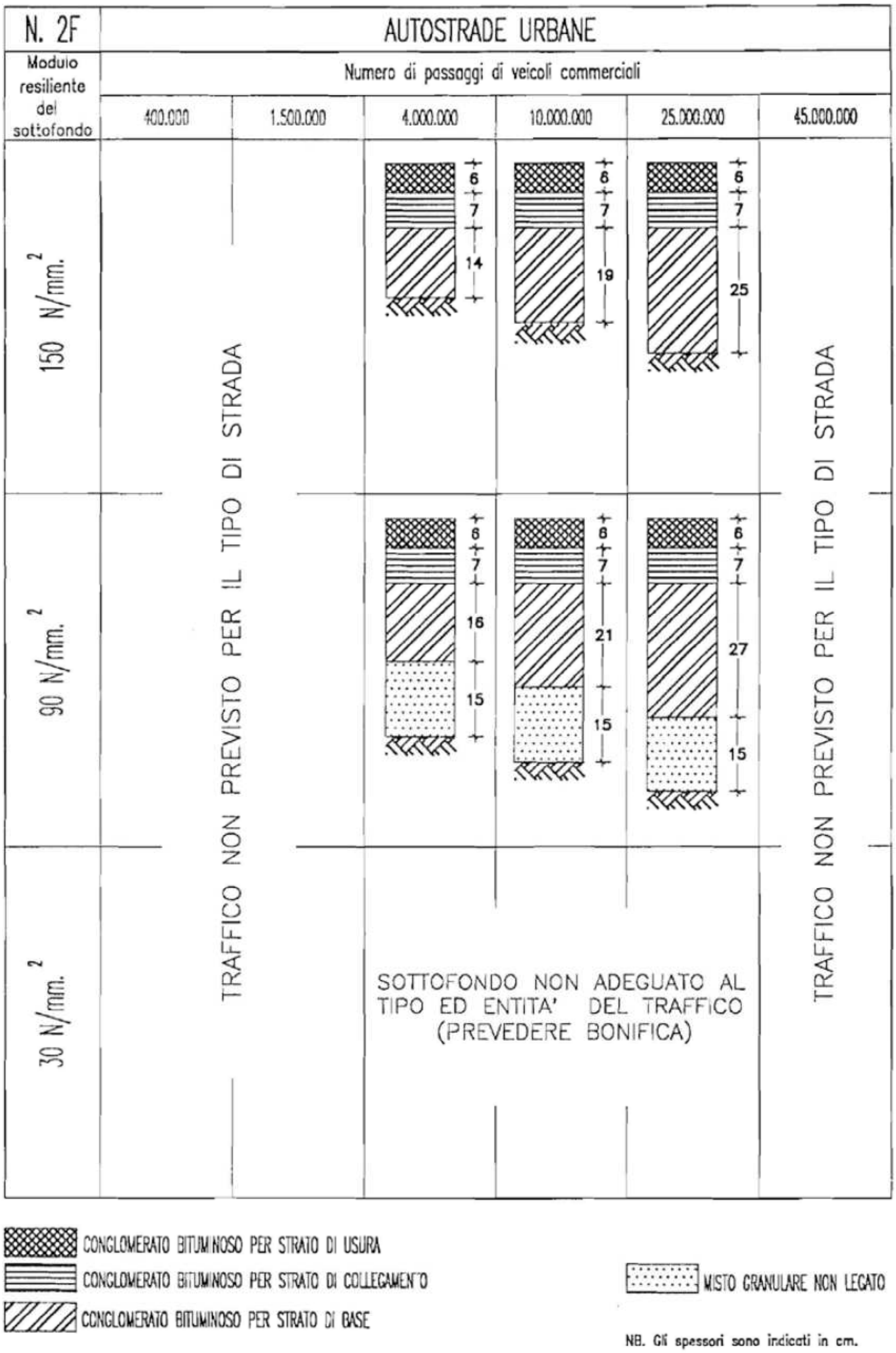


Figure 3 Italian Catalogue for Urban Roads with Flexible Pavement Design

The granular aggregates to be used for the base layer must have a granular composition, shows in the Table 5, according to the grading of the chart and a percentage of bitumen between 4% and 5%.

The grading envelope used for the base layers is quite different from the one used for the G1 base layer, as you can see in the Figure 4, since this kind of infrastructures have an asphalt base layers and it is not made of granular aggregates. (Ministero)

Table 5 Aggregates Grading

Sieve size (mm)	**Percentage passing by mass**	
	Nominal maximum size of Aggregate (mm)	
40	4	4
30	80	100
25	70	95
15	45	70
10	35	60
5	25	50
2	20	35
0,4	6	20
0,18	4	14
0,075	4	4

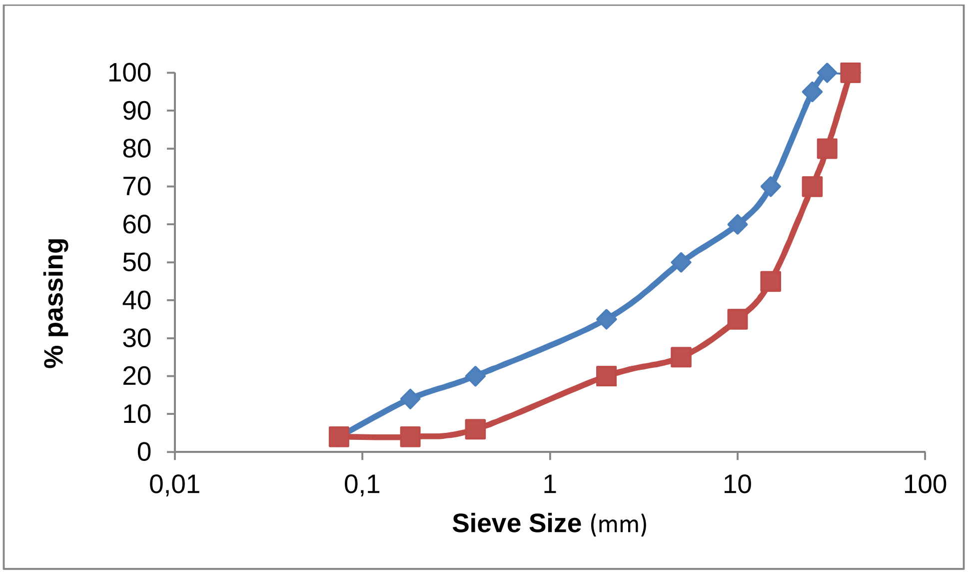


Figure 4 Grading for Hot Mix Asphalt Base Layer

2.5. Rigid Pavement

The element that marks a rigid paving is the presence of a base layer realized with concrete, that has the purpose of a surface layer and base layer and is laid on a sub-base with a cemented mix having the main purpose of creating a standard support plane. (Losa & al)

The rigid pavings are divided in various typologies:

- pavings with reinforced slab
- pavings with not reinforced slab
- pavings with continuous reinforced.

As shown in the Figure 5, rigid pavements generally consist of prepared roadbed underlying a layer of sub-base and base. The sub-base may be stabilized or unstabilized. In case of low volume road design where truck traffic is low, a sub-base layer may not be necessary between the prepared roadbed and the base.

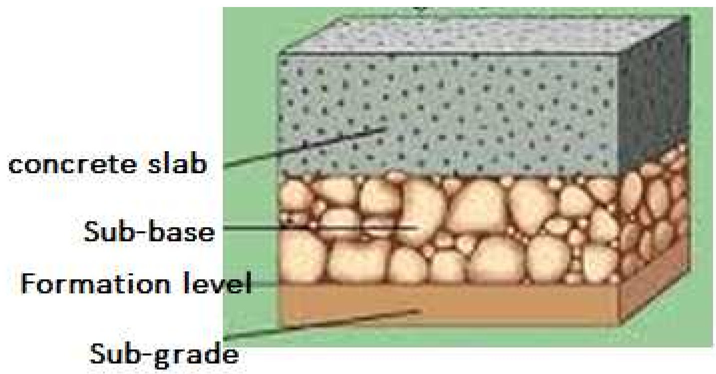


Figure 5 Rigid Pavement Scheme

The sub-base of a rigid pavement structure consists of one or more compacted layers of granular or stabilized material placed between the sub-grade and the base for following purposes:

- to provide uniform, stable and permanent support;
- to increase the modulus of sub-grade reaction (K);
- to minimize the damaging effects of frost action;
- to prevent pumping of fine-grained soils at joints, cracks and edges of the rigid base;
- to provide a working platform for construction equipment.

In the Figure 6 and in the Figure 7 we can notice the trend of the thickness of paving layers on varying the traffic entity and the category of road in the works.

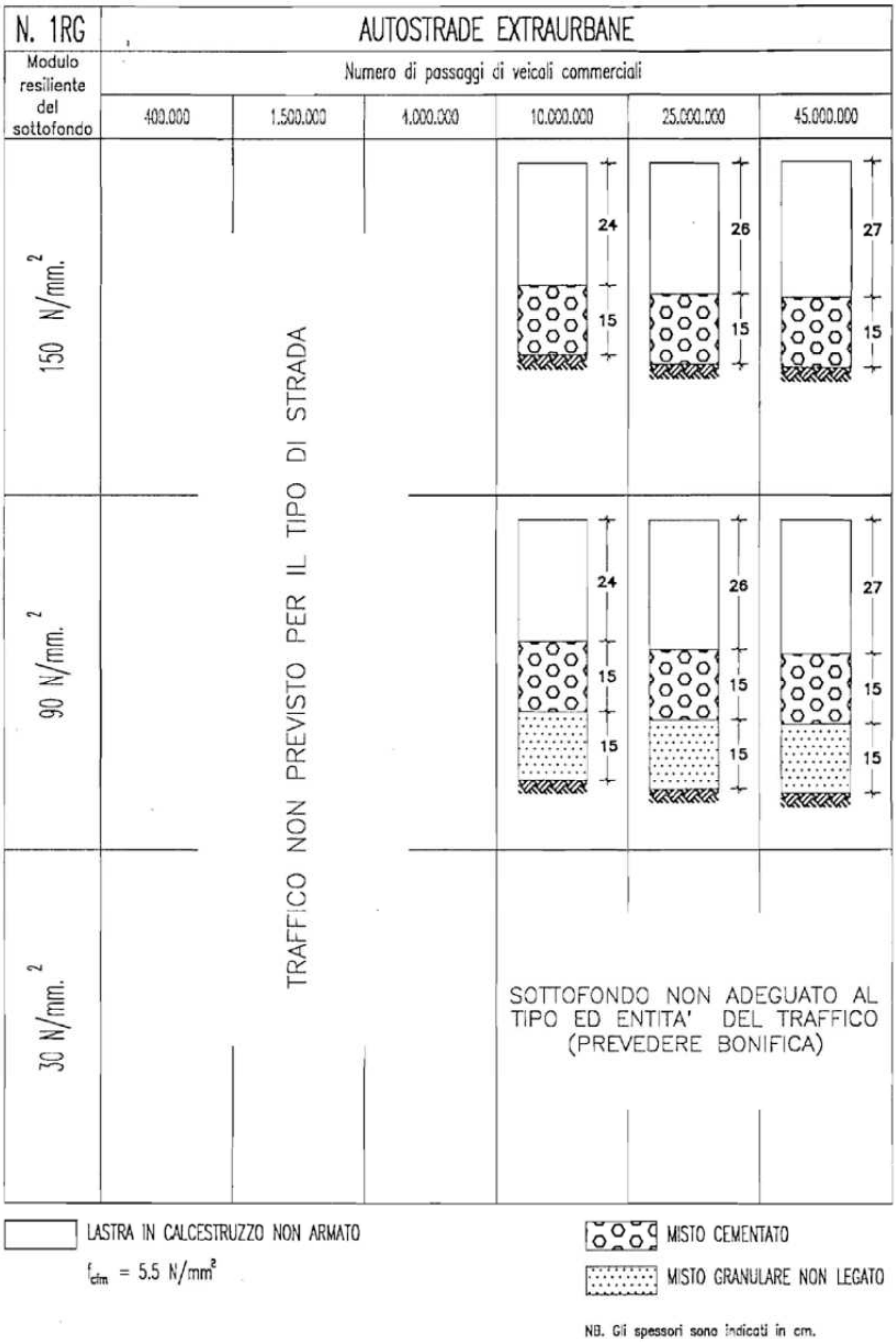


Figure 6 Italian Catalogue for Freeways with Rigid Pavement Design

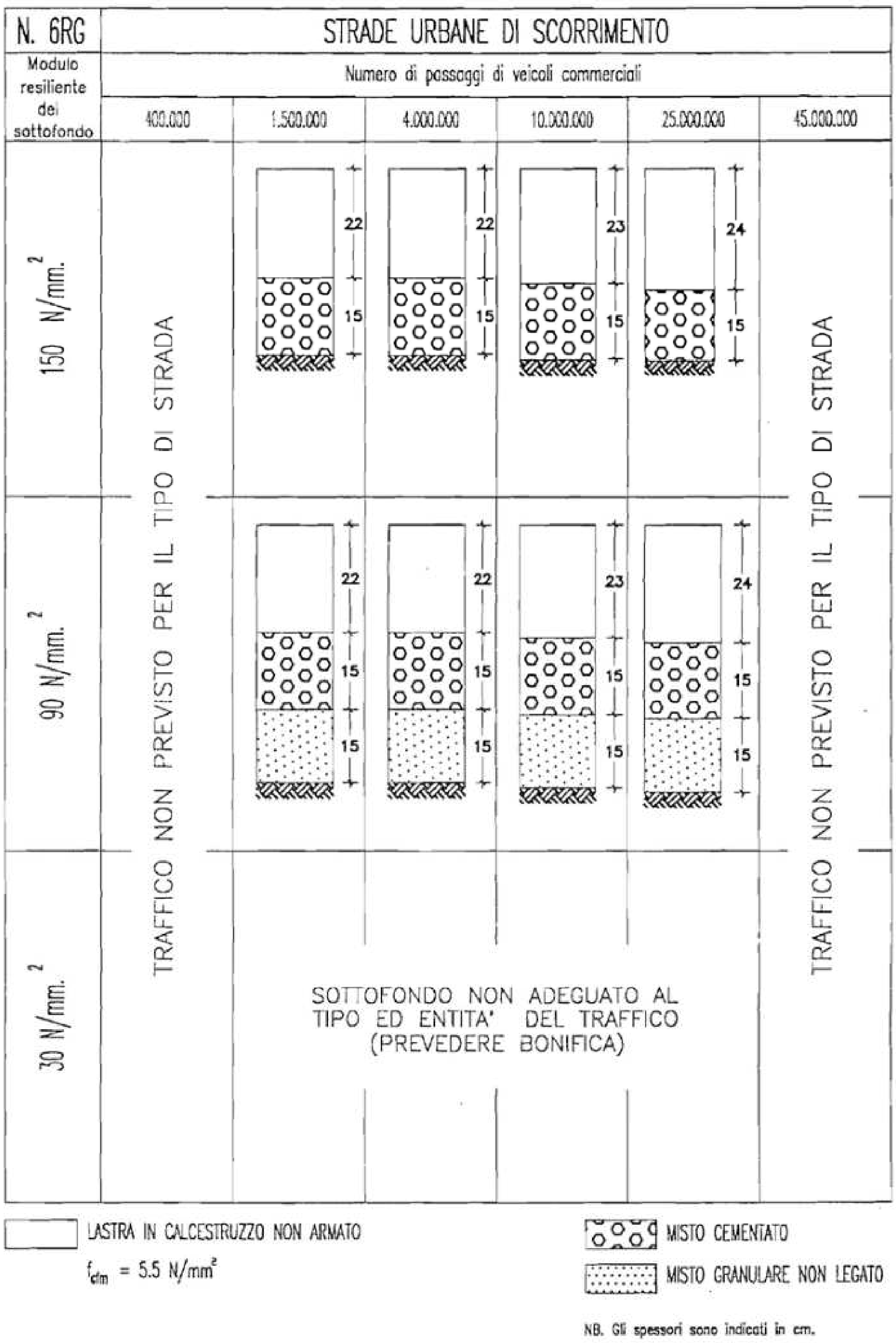
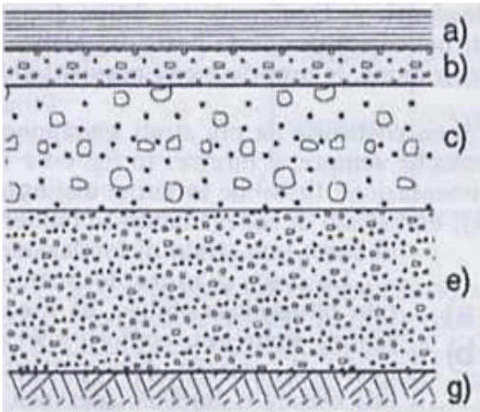


Figure 7 Italian Catalogue for Urban Road with Rigid Pavement Design

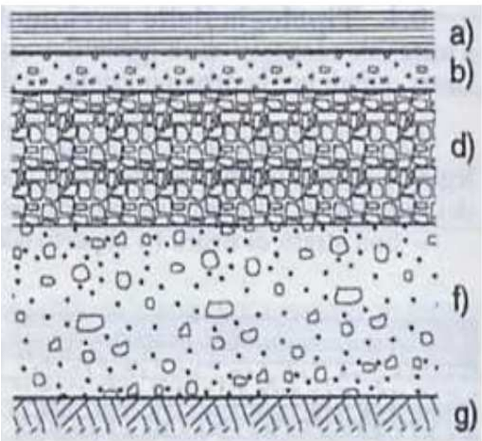
2.6. Semi Rigid Pavement

When in a flexible paving one of the bearing layers (base or foundation) is replaced, completely or partially, with a mixture of hydraulic components, it is possible to realize a semi-rigid paving. The semi-rigid pavings have a wide spread towards arterial roads characterized by strong amount of heavy traffic; indeed, the minor deformable condition of the rigid layer implies a minor deformable condition of the superstructure. Equal to other conditions, paving will felt less the effect of the effort and localized permanent deformations phenomena of the surface layers. In the Figure 8 and in the Figure 9 are reported two outlines of semi-rigid paving where the rigid layer is realized with a cemented mixture and it represents the base layer in the first case and the foundation layer in the second outline.



a) Surfacing (HMA)
b) Connection layer / binder (HMA)
c) Base (cement mix)
e) Foundation (coarse aggregate)
g) Subgrade

Figure 8 Semi rigid pavement scheme



a) Surfacing (HMA)
b) Connection layer / binder (HMA)
d) Base (HMA)
f) Foundation (cement mix)
g) Subgrade

Figure 9 Semi rigid pavement scheme

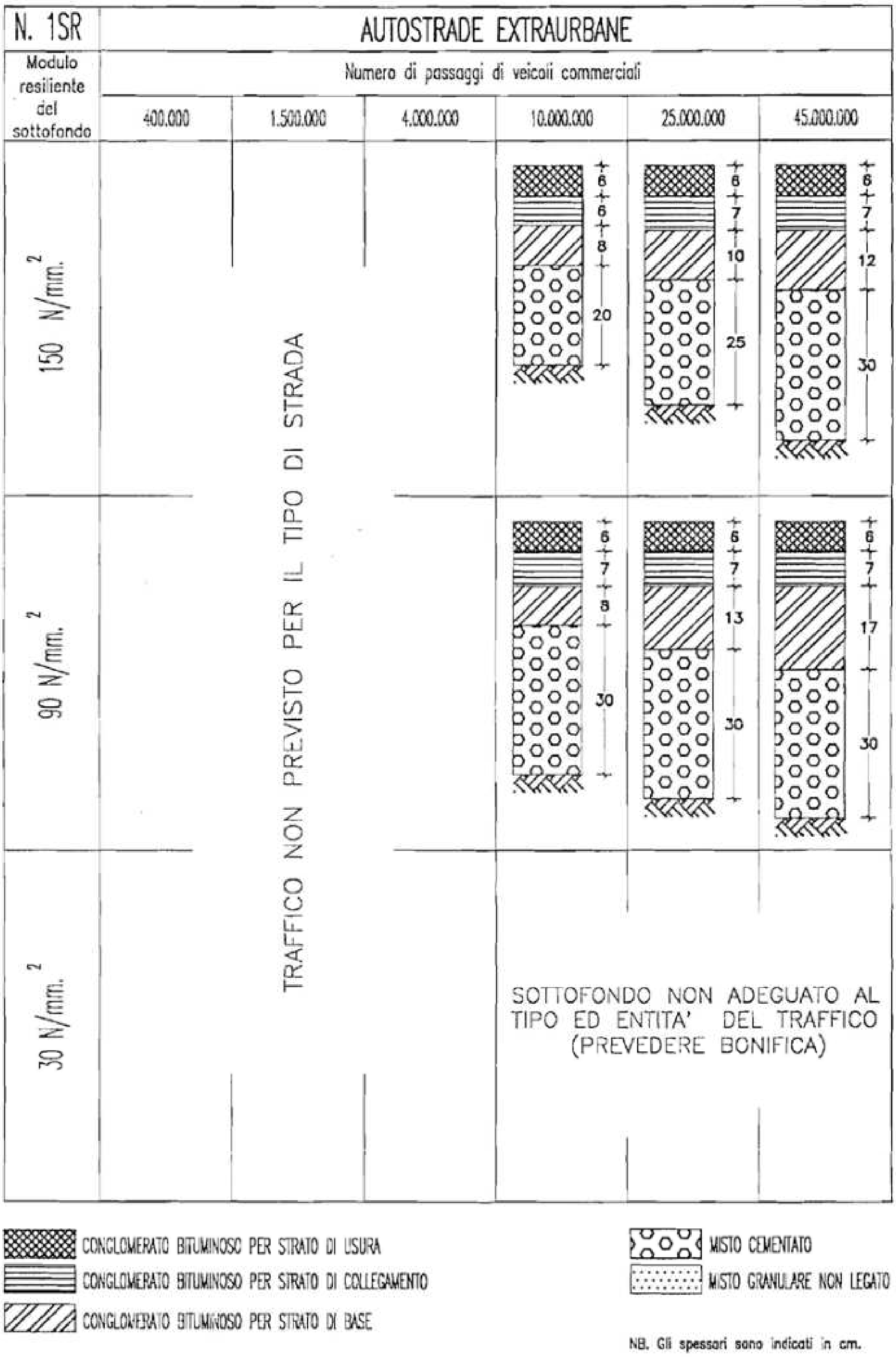


Figure 10 Italian Catalogue for Urban Road with Semi-Rigid Pavement Design

3. SOUTH AFRICAN OVERVIEW OF PAVEMENT DESIGN

In South Africa, the pioneer road-builder was Thomas Bain (1830 – 1893), son of Andrew Geddes Bain. Thomas Bain constructed 23 major mountain passes, nearly all in the Cape Province. Some of his roads are still in use today. Prior to the early 1920s, the thickness of pavement layers was based purely on experience. The invention of the car and the introduction by Henry Ford of his Model T-Ford in 1908 gave a strong impetus to look at the design of roads more seriously. Twenty million Model T Fords were sold between 1908 and 1927. Unpaved roads could not cater for this.

Thanks to the South African Pavement Engineering Manual (SAPEM) we can understand all aspects of South African pavement engineering (Agency, 2013).

One of the fundamental concepts used in pavement engineering is the nature of materials included in the layers. Two primary classifications of materials can be made:

Unbound Materials: This includes graded crushed stone, natural and crushed gravels, sand and soils. As traffic loads are applied, these granular materials interact with the layer beneath and respond with a stiffness that defines the extent of load spreading (stress distribution) in the structure. Only modest stress distribution is possible with unbound materials, given the moderate stiffness of their response. Repeated loads lead to an accumulation of deformation.

Bound Materials: This includes hot mix or warm mix asphalt, concrete, and cemented layers amongst others, which incorporate binders that “glue” the particles together. The materials have higher stiffness that results in flexural bending under load, and wide stress distribution. The bending beam effect results in the generation of significant tensile stresses in the layer, leading to damage in the form of cracking.

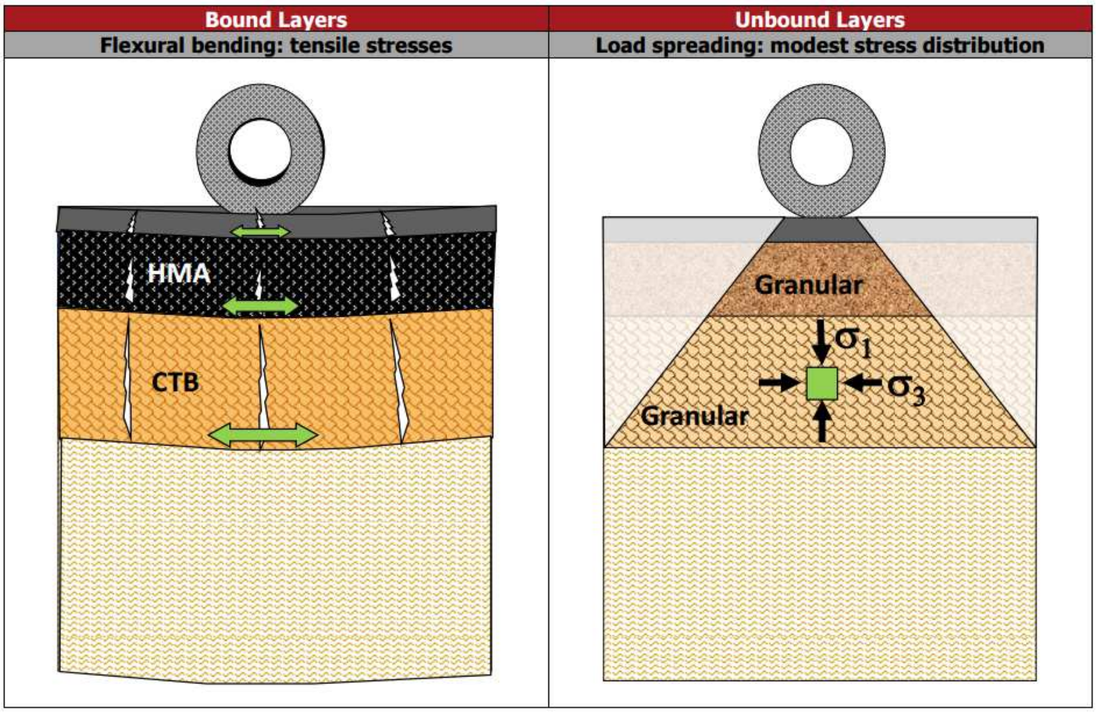


Figure 11 Two Categories of Pavement Materials and their Response to Loading

The pavement structure is the combination of layers and sub-grade, which carries the traffic loads. Typical pavement layered structures and names for each layer used in contemporary road construction are shown in the Figure 12 for flexible and rigid pavements.

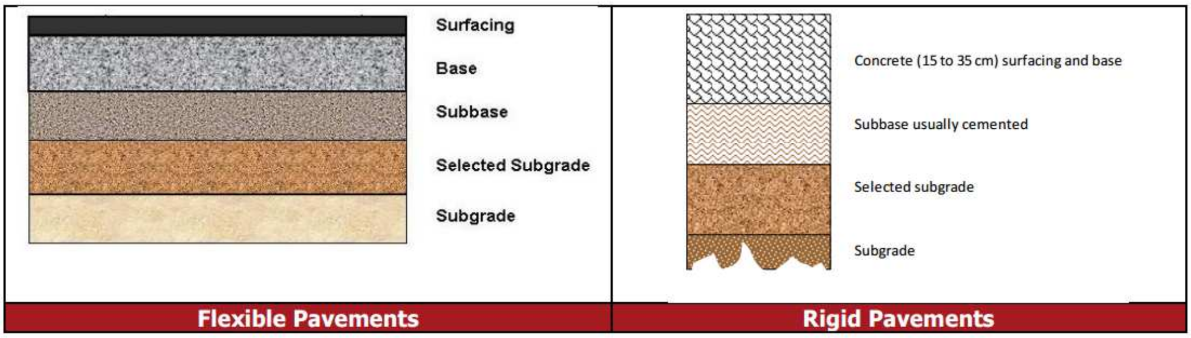


Figure 12 Typical Pavement Structures

The purposes of the various layers in the pavement are described below:

- Surfacing: This is a functional wearing course that provides waterproofing, skid resistance, noise-damping, durability against the elements, visibility and drainage. For surfaced roads, the upper layer is bound, consisting of spray seals, asphalt or concrete.
- Base: This is a load spreading layer and is the most important structural component of the pavement. The layer must provide the required support for the surfacing and distribute the very high tire pressures and wheel loads uniformly over the underlying

layers and sub-grade. The base comprises bound material, e.g., asphalt, concrete or stabilized, or it can be unbound, e.g., crushed stone or gravel base.

- Sub-base: This layer provides support for the base as well as a platform upon which to construct a structural base layer of high integrity. It also protects the underlying selected sub-grade layer by further spreading the load.
- Selected sub-grade: These layers are primarily capping for the sub-grade to provide a workable platform on which to construct the imported pavement layers. At the same time, these layers provide depth of cover over the sub-grade to reduce the stresses in the sub-grade to acceptable levels.
- Sub-grade: This is the existing material upon which the pavement must be constructed. It can be modified with stabilizers to reduce plasticity, ripped and recompacted to achieve uniform support, or undercut and replaced, depending on its quality.

Typically, the higher up the layer is in the pavement structure, the more expensive material to obtain or manufacture. Asphalt and concrete surfacing layers in a pavement are generally the most expensive layer in the pavement structure. It is also typical for the stiffer pavement layers to be at the top of the pavement structure. The exception to this is “inverted” flexible pavements, where the sub-base layer is cement stabilized and the base layer is a good quality granular layer. These pavements are widely used in South Africa.

Road pavement types can be classified according to the type of materials used to construct the upper layers, in particular the surfacing. An overview of different pavement types is provided in the Figure 13.

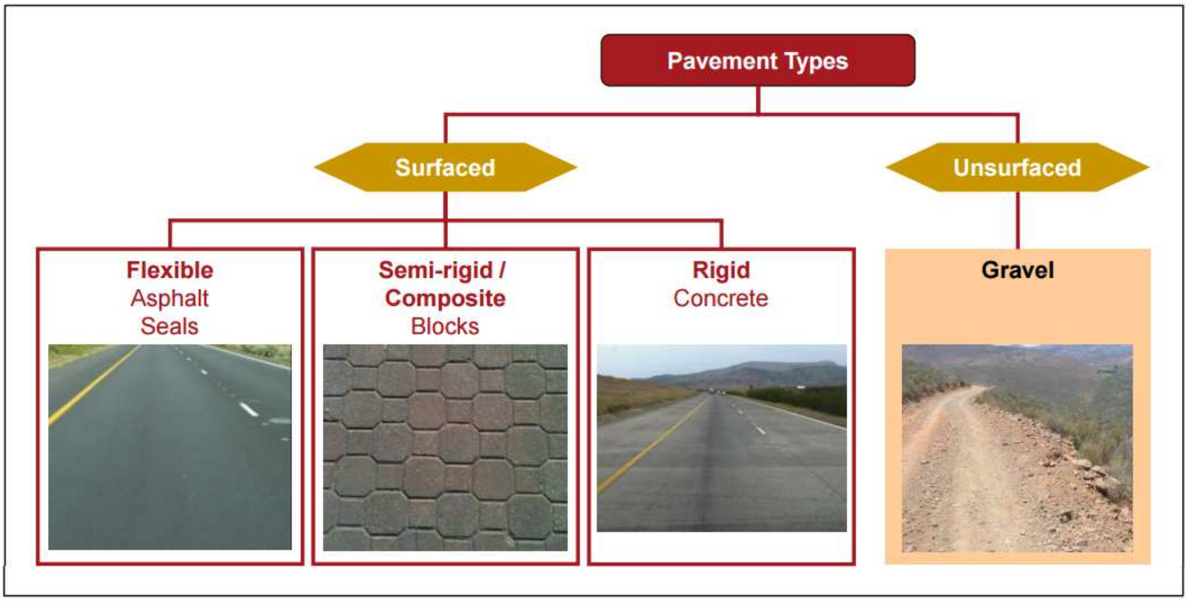


Figure 13 Classification of Pavement Types based on Materials

3.1. Road Classifications Based on Function and Traffic Levels

In addition to pavement types differentiated on the materials in the upper pavement layers, they are also classified according to their applications and levels of traffic, as summarized in the Table 6.

Table 6 Classification of Pavements Based on Application and Traffic

Facility	Traffic Class[1]	Loading
Freeway	Heavy, 30 to 100 MESA[2]	Light and heavy vehicles
Arterial and Main Road	Medium > 3 MESA	Light and heavy vehicles
Secondary Road	Light > 0.3 MESA	Low percentage heavy vehicles
Low Volume Road	LVR = 50 to 200 vpd[3]	Mainly light vehicles

[1] Traffic Class is also defined in TRH4 according to the upper limit of Equivalent Standard Axles (ES), e.g., ES100 = 30 to 100 million 80 kN axles.

[2] MESA = million equivalent standard axles (80 kN is the standard in South Africa, even though the maximum legal axle mass is 90 kN).

[3] vpd = vehicles per day

Roads and their related pavements are also be classified based on the importance of their function and the importance of the user trips made on the road. Functional classification is used to differentiate the minimum service levels for each class of road to set intervention levels with related budgetary implications. This functional classification is:

Class 1, Primary Arterials: High mobility between important cities, countries and transport hubs. Class 2, Secondary Arterials: Mobility links between slightly less important centers or connections to the primary road network.

Class 3, Minor Arterials: Connections between districts centers or between these centers and the primary and secondary road network.

Class 4, Collectors: Provide connections to the higher order network.

Class 5, Access Roads: Provide access to individual properties.

3.2. South African Flexible Pavement

The basic principles of structural design of a flexible pavement are explained using Figure 14, which shows a multilayered pavement system loaded by a dual-wheel, half-axle load. The stresses imposed by the tires at the tire pavement interface, also called the tire-pavement contact stress, are applied to a relatively small contact area. These stresses are dissipated or spread over an area that increases with increasing depth in the pavement structure. The stress concentration and shear stress, therefore, reduce with increasing depth. This is indicated by the red/yellow shaded area. The materials in the upper region of the pavement structure therefore need high shear strength to resist the imposed shear stress conditions. Deeper down in the pavement structure, less shear strength is required.

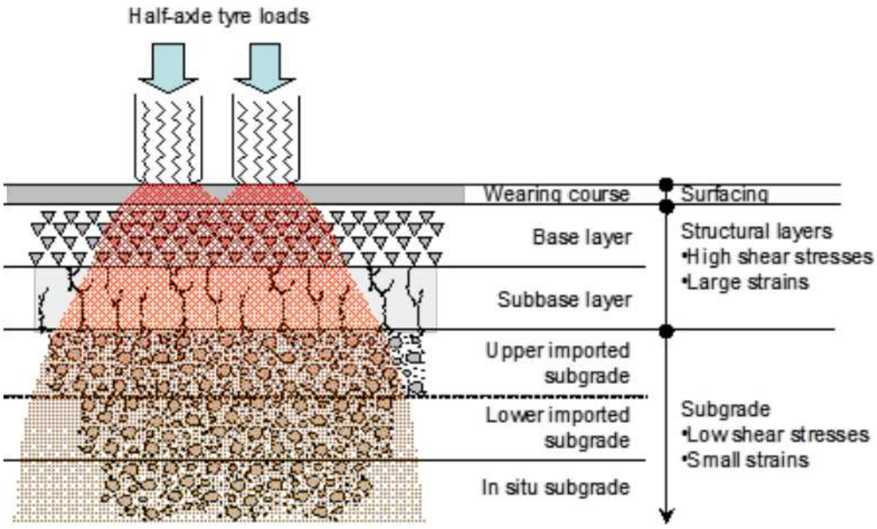


Figure 14 Stress Distribution in a Typical South African Flexible Pavement

In the South African infrastructures the bearing capacity totally depends on the base and the sub-base layers. The only performance required to the Asphalt Surface Layer is to guarantee an even surface and to allow adherence to the vehicles in transit. In South Africa, typical asphalt surfacing are between 30 and 50 mm thick. This type of infrastructure is realized for big traffic arteries. The success of this kind of paving is due to the high quality of the crushed rocks used for the base and the sub-base layers, particularly for the high level of compaction. For the sub-grade layer is required a minimum CBR rate of 15% and if this value cannot be reached it is necessary to stabilize the sub-grade layer.

The South African pavements are usually made of a surface layer, a base layer, a sub-base layer and a sub-grade layer.

<u>Sub-grade:</u>

It is the completed earthworks within the road prism before the construction of the pavement. This comprises the in-situ material of the roadbed and any fill material.

<u>Selected Layer</u>:

It is the lowest layer of the pavement consisting of controlled material, either in situ or imported.

<u>Sub-base</u>:

It is the layer(s) occurring beneath the base or concrete slab and above the selected layer(s).

<u>Base</u>:

It is the layer(s) occurring immediately beneath the surfacing and above the sub-base, above the selected layer(s).

<u>Surfacing</u>:

The uppermost pavement layer, which provides the riding surface for vehicles.

As you can see in the Figure 15 below:

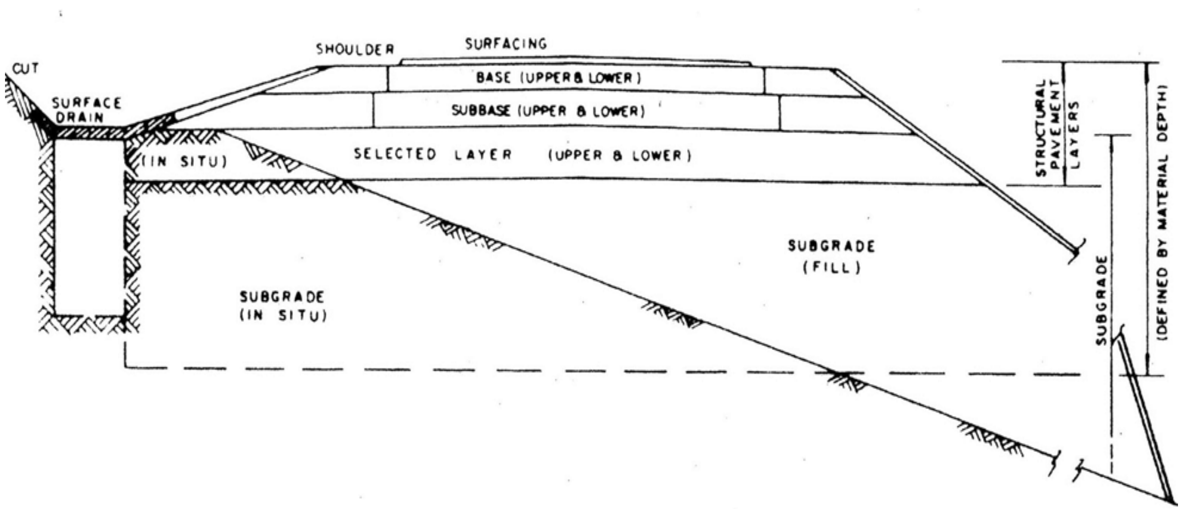


Figure 15 Scheme of Road Infrastructure

Asphalt surfacing provide the interface between the tires of vehicles and the pavement, and are, therefore, one of the main structural layers of the pavement. They should meet the engineering properties and should be textured for adequate skid resistance. The following asphalt surfacing are generally used:

- Gap-graded (AG)
- Continuously graded (AC)
- Semi-gap-graded (AS)
- Open graded (AO)
- Stone mastic asphalt (SMA)
- Semi-open graded asphalt (ASO)
- Ultra-thin friction course (UTFC)

Asphalt surfacing, or wearing coarse, can be divided into two broad categories in terms of their primary purpose:

Structural layers generally have a specified thickness of more than 30 mm. They are designed to contribute measurably to the strength of the pavement and to provide adequate skid resistance for the prevailing traffic and climate conditions.

Functional layers have a specified thickness of less than 30 mm and do not contribute significantly to pavement strength.

They are best described as surface dressings that meet functional criteria such as:

- Suitable surface texture for skid resistance, noise reduction and surface water drainage, given the traffic volumes, speed and prevailing climate.
- Sealing of the substratum against water penetration.
- Limited improvement of riding quality

3.3. Types of Bases and Sub-bases

There are various types of bases and sub-bases typically used in pavement structures.

- Granular layers range from well graded natural gravels to crushed gravels and crushed stone bases. G1 crushed stone bases are a unique form of crushed stone base, and have specific grading and construction– methods to ensure the layers have a high density.
- Water bound macadam is another unique type of crushed stone base. It consists of large particles of stone (37 mm) in a matrix of non-plastic sand that is washed and vibrated in between the large stones with large amounts of water.
- Bituminous bases normally have large aggregates and bituminous binders that are less moisture susceptible than granular materials.
- Bituminous Stabilized Materials (BSM) are gravels and crushed stone materials stabilized with bitumen and small amounts of cement or lime, to provide some strength but primarily to reduce moisture susceptibility, while retaining more flexibility than a purely cemented layer.

3.4. South African Inverted Flexible Pavement

An inverted base pavement is a pavement structure that consists of an unbound aggregate base between a stiff cement-treated foundation layer and a thin asphalt surface layer. An inverted pavement is when the base layer is a high quality granular layer, and the sub-base a cement stabilized layer. A thin asphalt layer or seal provides the surfacing. The term “inverted” is used because the strength of the pavement does not decrease with pavement depth, because of the stiff cemented layer. This means that the pavement is not in balance. The idea behind an inverted pavement is that the cemented layer provides an anvil upon which the granular base can be well compacted with an additional compaction called Slushing Process. This achieves a high quality, dense base. Over time, the cemented layer weakens to an equivalent granular state. The pavement is then in balance. Inverted or “upside-down” pavement structures are commonly used in South Africa, and are included in the TRH4 catalogues. (SANRAL, 2014)
The combination of the layers in the inverted pavements base can reduce the dependency of pavement construction on asphalt, while, at the same time, allows a high quality pavement for all traffic levels by using the granular base as structural key element (Tutumluer E. , 2013). In fact unlike conventional pavements which rely on upper stiff layers to bear and spread traffic loads, the unbound aggregate inter-layer plays, in an inverted base pavement, a major role in the mechanical response of the pavement structure (Cortes, Shin, & and Santamarina, 2012).
This kind of paving was developed in South-Africa by seeking to obtain the maximum performance by the unbound aggregate and to reduce the use of hot mix asphalt. The Inverted pavement structure now is commonly used in South Africa to support heavy traffic load and validated through many years of Heavy Vehicle Simulator. From 1950 there has been a big development since a couple of engineers have deepened studies concerning the layer compaction in UAB and experimental test sections (Jooste & Sampson, 2005).

3.5. Slushing Process

The need to obtain civil engineering structure stable and durable is tightly rely to the capacity to compact the granular materials as much as possible. This is always possible, but the energy consumption should be commensurate with sustainable costs for each work. The effectiveness of this construction technique is largely entrusted to the bearing capacity of the base layer made with mixed granular unbound. The excellent performance of this base is achieved through a high level of compaction, enhanced by an additional "slushing" operation. For this reason rock properties and grading are necessary to obtain great results by the slushing process of the base layer; precise and sequentially slushing process were analyzed since are the major factor that lead a successful conclusion of the Inverted Base Pavement structures.

The pavement response improves when the density of the material, and consequently its stiffness, is increased. Furthermore, because of the better load spreading ability of well compacted material, the same protection to the sub-grade can be provided by a less thick pavement layers when these have a high stiffness (i.e. high density). The pavement structure can therefore be more economical and less demanding in terms of natural resources when the materials used are well compacted. South Africa unbound granular base layers are named with the acronyms of G1-G10, from high to poor mechanical performances, by the South African Technical Recommendations for Highways (Structural Design of Flexible Pavement fon Interurban and rural roads, 1996). It is possible to obtain the G1 layer only when we have a good aggregates and if we apply slushing process.

The G1 crushed stone base layer is probably one of the vital layers in modern day high quality pavements in South Africa. Depending on the production rate, the cost of compaction only makes up approximately 10% of the total cost for a G1 quality base course, At this relatively low cost the density of the G1 material is increased from approximately 70% ARD to 88% ARD, which corresponds to a seven fold increase in the strength (CBR) of the material. This increased strength generally leads to increased pavement life and it can thus be deduced that a relative low investment in good compaction (high density) returns high value in terms of extended pavement life (Ebles, Lorio, & Van der Merwe, 2004).

The geology of base course aggregates is an important aspect which can influence the performance of aggregates in pavement and the engineering properties of aggregates can be assessed if the type of the source rock is known as the rocks are classified based on their mineralogy, grain size, and texture. The high stiffness and durability of the base layer of the infrastructure is obtained thanks to the use of a rock with excellent physical-mechanical performance. The type of the rock along with a petrographic description provides a sound foundation to judge the engineering properties of the source rocks. The type of the source rocks can be identified by the microscopic investigation of the rock by making thin sections (Higgins, 2000). Thin sections can be inspected under the electron microscope to identify various types of minerals present in the rock. The strength of aggregates is dependent on the shape of grains, arrangement of grains, and the cementations nature of the abundant mineral found in the matrix of the aggregate.

3.5.1. Sub-Base Layer Surface Before Slushing

The Sub-base layer on which the G1 layer is built must have been previously cleaned, slightly wet and levelled, because great energies of compaction and copious quantities of water will be deposited. Consequently the sub-base layer must be of an excellent quality. Having an irregular or rough sub-base surface facilitates the segregation of the aggregates during construction and it also affects the "slushing process". If the surface of the sub-base layer proves poor quality, it may be unable to perform the slushing. To obtain a stable support during compaction and a good resistance associated to the abundant addition of water in the "slushing process", the sub-base layer must be cemented with C4 or C3 type as per Technical Recommendation for Highway (750-1500 kPa UCS) (TRH4, 1996).

3.5.2. Prepare G1 Base Layer For Slushing

The G1 layer should be compacted using heavy pneumatic-tired roller (over 17 tons.) in combination with the static steel-tired rollers or Three-Wheeled roller as shown in Figure 16 one behind the other. It is extremely important to drive the roller machines for compaction with special attention and under supervision of an employee, which will control especially the first stages of compactions to prevent the so-called "bow wave" of material in front of the roller-drum.

Figure 16 Steel-tyred rollers and Three-Wheeled pneumatic roller

The compaction continues until the layer shows no movement to the passage of heavy rollers above it.

At this point of realization, the density of G1material should be around 85% of SRD / ARD (102%Mod).

Only after this condition the "slushing process” can be started, because if it starts too early the layer would become unstable and it would expel some of the larger fines, making this process long and complicated.

3.5.3. Slushing Procedures

During this process the layer will be subject to additional rolling in order to obtain a firm, even, well-knit surface. Even under favourable warm and windy conditions, at least two days will be required after compaction of the final base layer for it to dry out sufficiently and “set” prior start the slushing.

After the base has been compacted to the required density at optimum moisture content (OMC) and it has been allowed to dry out to less than 50 percent of the OMC, the slushing can commence (WES-KAAP, 2008).

The slushing process starts by wetting sections of 40 to 60 meters layer at a time (depending on the number of rollers available).The Figure 17 shown the begging of the slushing process when the water must be applied by using a "tank truck" at the highest points of the cross section or gradient because it runs down to lower points. It is difficult and sometimes it may be incorrect quantify the optimum quantity of water, because it will vary greatly with the type of material, dryness, temperature, and humidity (AFM, 1968).

Figure 17 The tank truck at the begging of Slushing Process

Right afterward this section is rolled with the Heavy Static Rollers and pneumatic roller (minimum 12 tons) (COLTO, 1998).

Keep in mind that the use of relatively light rollers slushes only the upper part of the layer. The Pneumatic-Tired Roller, in the Figure 18 is a machine suitable for the compaction of granular materials, unlike the static roller, the presence of the tires and their deformation to the contact of the soil generates an effect "kneading" able to make numerous benefits.

Figure 18 The Pneumatic-Tyred Roller

The steel wheel roller helps to get an even and less rocky surface whereas the kneading action of the pneumatic roller helps especially the initial rolling to even-out the bladed surface (Senadheera & Vignarajah, 2007). In fact, the rubber roller transmits to the layer either a vertical force related to the mass of the roller and horizontal forces related to the stresses tangential due to the deformation of the tire.

During this process the minus 0,075mm fraction is used as a “lubricant” to ease the relative movement between the larger particles towards achieving intimate stone upon stone packing, squeezing (slushing) the excess fines out of the matrix in the process. It was found advantageous having the percentage minus 0.075mm fraction slightly higher (2% - 3%) than required in order to satisfy the Fuller curve grading, especially when working with material which is on the coarser side of the specification envelope. (Kleyn E., 2012).

During Watering and rolling procedures the slightly excess of fine will tend to go out on the surface. By using a hand or a mechanical brooms we must distribute over the surface of the base and eventually off the road the excess of fine without creating a skin of it. Any areas deficient in fines shall be corrected by adding fine aggregate by the same source of supply as the crushed aggregate.

During the process, developed suction forces will contribute to the final strength of the material. (FHWA, 1997).

It was also observed that air bubbles, shows in the Figure 19, which appear on the surface indicate that the aggregate is moved closer, expelling the air from the voids of the matrix and the fine particles in excess.

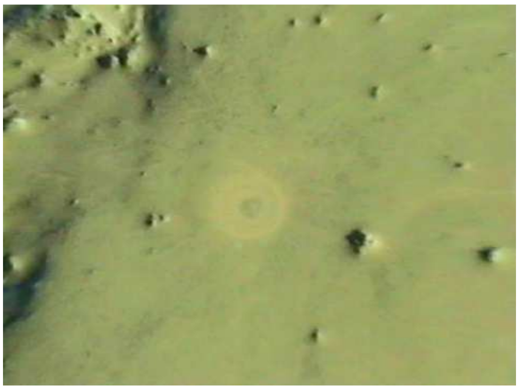


Figure 19 The air bubbles on the surface of the layer

This phenomenon will end during the final stages of the process when a solid and stable matrix without voids will form. A sign that the slushing process has been completed and should be stopped is when just clear water is expelled. Finally, all slush-fines must be removed from the road with heavy duty hand brooms or light mechanical brooms before it dries out (and hardens to a crust) as we can see in the Figure 20 If you do not care at this indicator, sandy textured fines (the fraction above the minus 0.075mm) usually starts being expelled.

Figure 20 Slush-fines removed from the road with heavy duty

The slushing process increases the overall Apparent Relative Density (ARD) by 2% - 4% , this means that at this point 88% of ARD is achieved, (equivalent to about 106% Mod AASHTO density) with an improvement of a shear strength and performance of the material.

The layer should be now allowed to dry out somewhat for about 12 hours and no traffic should be allowed on it. After that, the layer can receive its final "roll dry" using a steel-wheeled roller in order to improved the surface, because even if not visible, the aggregated particles will be slightly raised due the excess of water around and under the coarse aggregate during the drying which should now be evaporated. The Figure 21 shows the final “roll dry”.

Figure 21 Final "roll dry" using a steel-wheeled roller.

The completed layer shall be firm and stable with a closely knit surface aggregate exposed in mosaic and free of areas of segregated material, laminations or corrugations.
The layer shall be primed as soon as the moisture content of this layer is below 50 percent of optimum moisture content to protect the base until the surface asphalt is applied. Immediately before priming the base shall be lightly brushed with a rotary broom to remove any dust and to lose material to ensure adhesion between the layers.
Now that this process is ending, the aggregate is interlocked very well. Unfortunately, by applying more energy, the only result is the fracturing of the aggregates, creating more fines and hence, degrading the interlock and shear strength of the G1 matrix.
This slushing must be carried out on each section in one continuous process, and each section shall be completed before starting with the next one.

3.5.4. Quality Control

Quality control is essential to any project success. Although visual inspection is important, unfortunately, it is not enough to control the construction layers which contain considerable fine material. It was observed in fact that incomplete removal of fines to expose good mosaic after slushing and before sealing can cause a loose or weak layer at the top of a completed pavement layer (Netterberg & De Beer, 2012). Control tests will include determinations of gradation, mixing proportions, plasticity characteristics, moisture content, and field density.
Density/quality control should be done within 24 hours when the material is still damp (moisture content of about 50% OMC in the upper portion of the layer) and the possibility of disturbance minimized. COLTO adopted density control by nuclear apparatus, mainly because the possibility of disturbing the compacted aggregate matrix is less than excavation.

Slushing process is a technique of compaction used for more than 25 years in South Africa that aims to obtain high values of compaction. This is possible only if the grading is the correct one and the rock is of excellent quality.

The slushing process removes excess fines used for lubrication during compaction and compresses the coarser particles in contact with each other brings many benefits in the layer as a decrease in the risk of deformation, increase internal friction, decrease void index and resulting in improved impermeability. Only performing properly and carefully all steps can be realized a successful layer.

3.6. South African Rigid Pavement

The principles for rigid pavement design are those according to which the transition from material with high shear strength and stiffness at the top to lower shear strength and stiffness material in the sub-grade is rapid, not gradual. The primary load supporting element of a concrete pavement is the rigid layer or concrete slab. The shear strength and stiffness of concrete is high in relation to asphalt or crushed stone road bases, and the imposed stresses are dissipated quickly in the rigid layer. A thin layer of concrete thus protects the sub-grade in a similar way as thicker layers and combinations of asphalt, crushed stone and gravel materials. As you can see in the Figure 22.

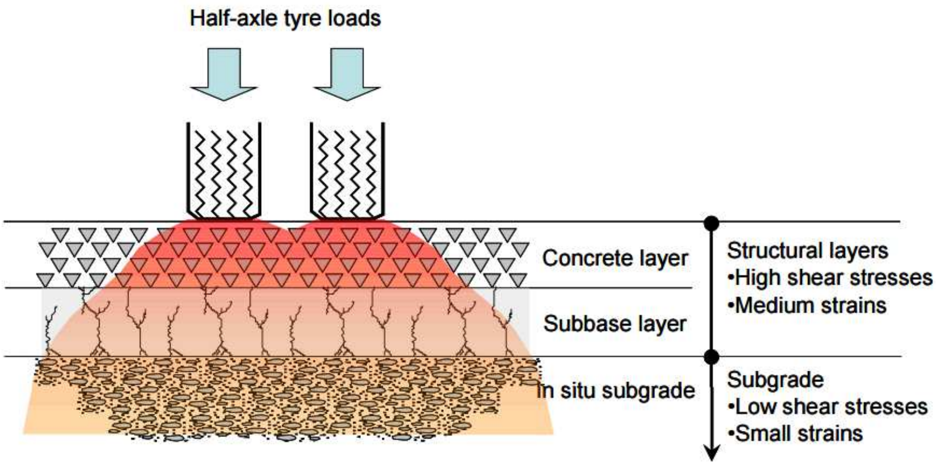


Figure 22 Typical Stress Distribution in a Rigid Pavement

The essential elements of concrete pavement design are to design the slab length, slab thickness, and sub-base support type. The slab length is important to mitigate shrinkage cracking. In Plain Jointed Concrete Pavements (PJCP), shrinkage cracking is controlled by providing joints at regular and relatively short intervals. Failures in concrete pavement generally occur at joints and cracks. Design, therefore, focuses on joints and cracks, with the aim of ensuring proper load transfer. Dowels are often installed at joints to improve load transfer across the joints and the concrete pavement is then referred to as a dowel jointed plain concrete pavement.

The following types of rigid pavement design, which differ only by the crack control criteria, are the most common concrete road pavements in South Africa.

- Jointed unreinforced (plain) concrete that can be doweled or not doweled
- Jointed reinforced concrete pavement with light reinforcement to increase joint spacing
- Continuously reinforced concrete pavement (CRCP)
- Ultra-thin concrete pavement (UTCP)

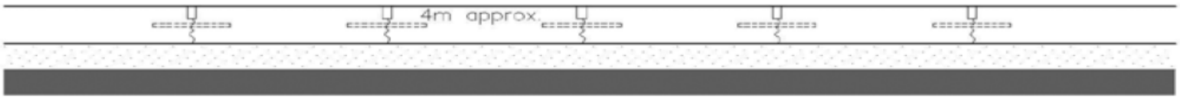


Figure 23 Jointed Unreinforced type of Rigid Pavement

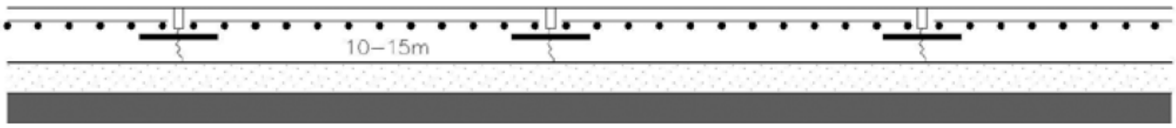


Figure 24 Jointed Reinforced Concrete Pavement

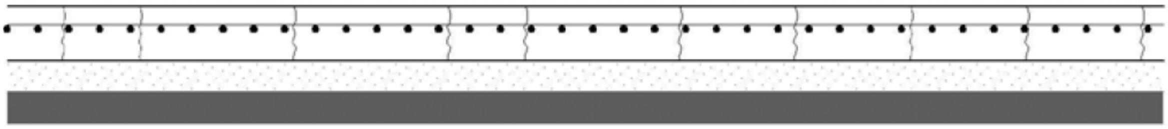


Figure 25 Continuously Reinforced Concrete Pavement

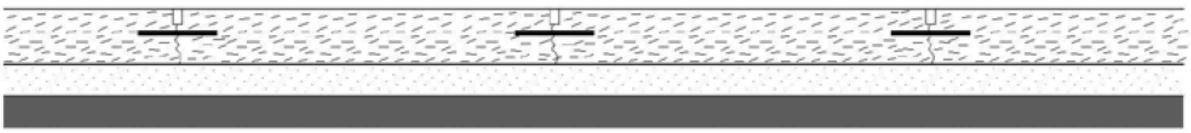


Figure 26 Fibre Reinforced Concrete Pavement

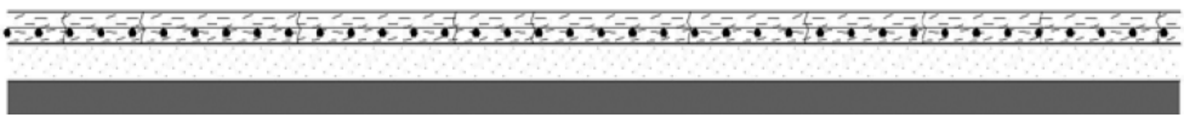


Figure 27 Ultra thin Continuously Reinforced Concrete Pavement

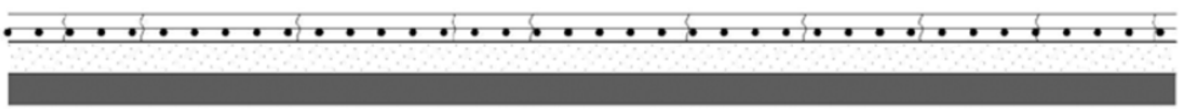


Figure 28 Ultra Thin Reinforced Concrete Pavement

The sub-base layer provides the primary support to the base, lower layers such as selected sub-grade layers in turn support the sub-base. The specifications for these layers depend primarily on the sub-grade conditions and need to provide adequate cover to the sub-grade to support the sub-base. The quality and strength gradually improves towards the surface, to provide a balanced structure.

3.7. Road Category

In South Africa, roads are categorized according to the importance and level of service required. The road authority has a number of road categories to suit the different levels of service the system has to deliver based on the associated service objectives.

The pavement for a Category A road will be normally constructed and maintained to higher functional standards (safety, riding quality, comfort, etc.) than pavements for Categories B, C and D roads, Table 7 and Figure 29. (TRH4, 1996)**.**

Table 7 Definition of the road category

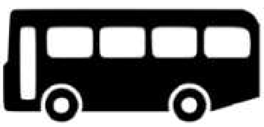
Road Category				
	A	B	C	D
Description	Major interurban freeways and major rural roads	Interurban collectors and rural roads	Lightly trafficked rural roads, strategic roads	Rural access roads
Importance	Very important	Important	Less important	Less important
Service Level	Very high level of service	High level of service	Moderate level of service	Moderate to low level of service

Figure 29 Typical South African Roads for Road Category

In South Africa, as in many other countries, the standard axle load is 80 kN. However, the legally permissible axle load is 88 kN. (TRH4, 1996)

The permissible axle load limits, in terms of the average static axle mass for different axle groups are summarized in the Table 8 (TRH 16).

Table 8 Permissible Axle Loads

Vehicle	Axle Group		Wheel Configuration	Permissible Static Mass (kg) Pre-1996[1]	Post-1996[2]
Truck	Steering		Single	7 700	7 700
	Single		Dual-wheel	8 200	9 000
			Single	7 700	8 000
	Tandem		Dual-wheel	16 400	18 000
			Single	15 400	16 000
	Tridem		Dual-wheel	21 000	24 000
			Single	21 000	24 000
Bus	Steering		Single	7 700	7 700
	Single		Dual-wheel	10 200	10 200

In this document pavement is designed to have a specific bearing capacity which is expressed in terms of the number of Standard (80 kN) Axle (SA) load repetitions that will result in a certain condition of deterioration. This condition is normally considered to be the *terminal condition*, indicating that the pavement has structurally "failed", and it cannot longer support the *functional service* set by the service objective.
A pavement could have a *bearing capacity* of *1* million Standard Axle repetitions (1 x 10^6 SAs), indicating that the pavement will be able to carry a *traffic spectrum* to the *Equivalent of* 1 *million Standard Axle loads (1 x* 10^6 ESA).
Thus:

- Pavement Bearing Capacity is expressed in Standard (80 kN) Axle repetitions (SAs or 80s), and
- Traffic Load Spectrum (i.e. traffic demand) is expressed in: Equivalent Standard Axle repetitions(ESAs or E80s).

For the purposes of the Pavement Design Catalogue, the pavements are divided into ten different classes, namely ES 003 to ES100, covering extremely light traffic to extremely heavy traffic. The classification is summarised in Table 9. For each of the ten pavement classes given, the design bearing capacity in terms of million standard 80 kN axles/lane (million SAs/lane) is also given. The volume of traffic for each pavement class is given separately. For pavement classes ES0,003 to ES3, the volume of traffic is based on vehicles per day per lane (v.p.d./lane), and for classes ES10 to ES100 the volume of traffic is based on vehicles per day per direction (v.p.d./direction). We have to convert the v.p.d to equivalent standard axles.
Pavement classes ES0,003 to ES0,3 usually provide for very light to extremely light traffic, and may include pavements in the "transition" from gravel to paved roads. This relatively finely divided group of pavements may incorporate semi-permanent and/or all weather surfacing, like gravel bonding agents, and is usually more weather sensitive than the group ES1 to ES100.
Pavement classes ES1 to ES100 provide for the lightly trafficked roads to very high volume and/or high proportion of fully laden heavy vehicles. These roads always incorporate an all weather good quality surfacing.

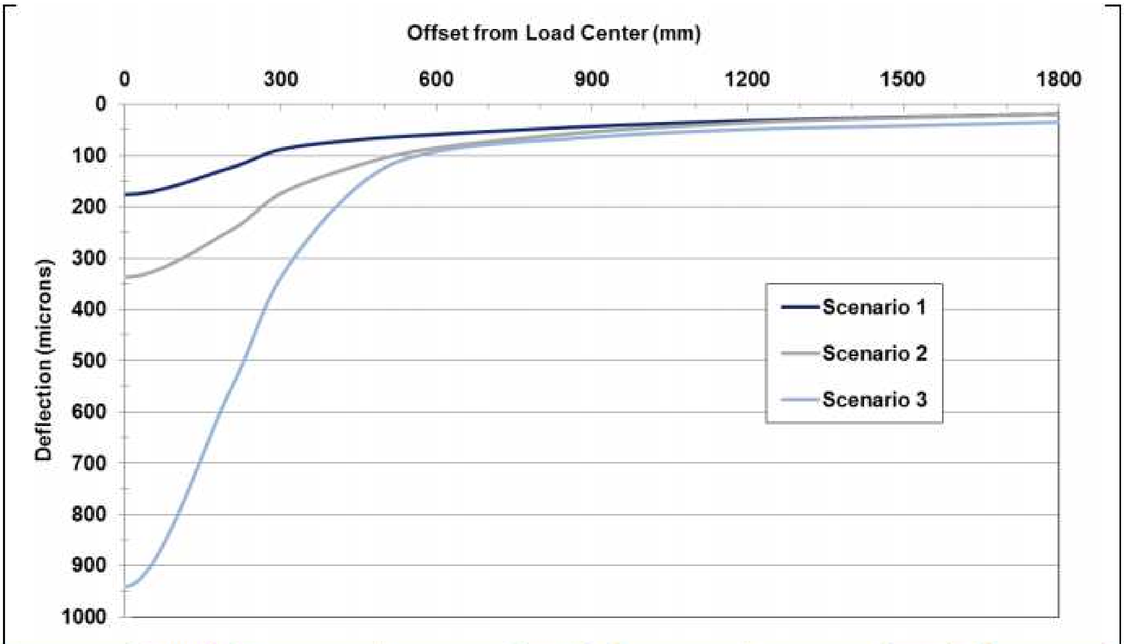
Table 9 Classification of pavements and traffic for structural design purposes

Pavement Class	Pavements design bearing capacity (million 80 kN axles/lane)	Volume and type of traffic	
		Approximate v.p.d. per lane	Description
ES 0,003	< 0,003	< 3	Very lightly trafficked roads; very few heavy vehicles. These roads could include the transition from grave to paved roads and may incorporate semi-permanent and/ or all weather surfacing
ES 0,01	0,003 - 0,01	3- 10	
ES 0,03	0,01 - 0,03	10 - 20	
ES 0,1	0,03 - 0,10	20 - 75	
ES 0,3	0,10 - 0,3	75 - 220	
ES 1	0,3 - 1	220 - 700	Lightly trafficked roads, mainly cars, light delivery and agriculture vehicle; very few heavy vehicles
ES 3	1 - 3	> 700	Medium volume of traffic; few heavy vehicles.
ES 10	3 - 10	> 700	High volume of traffic and /or many heavy vehicles.
ES 30	10 - 30	> 2200	Very high volume of traffic and/or a high proportion of fully laden heavy vehicles.
ES 100	30 - 100	> 6500	

3.8. Pavement Behaviour Under Loading

The effect of vehicle loading on a pavement is relatively small, when considering each vehicle or loading individually. However, the cumulative effect of many such loads causes distress in the pavement. An understanding of the short term effect of loading on a pavement provides a good background for how the cumulative affects manifest and are modeled. Under the action of a moving vehicle load, the pavement deflects and rebounds when the load has moved away. The effect of a heavy vehicle load generally extends over an area of 1 to 2 meters from the point of loading, in all three directions. This deflected area tends to form a circular, deflected indentation known as a deflection bow. The size and shape of deflections bow vary and depend on the pavement structure, the strength and stiffness of the materials, pavement balance, temperature and of course, the loading magnitude, duration and contact area. For flexible pavements in a good condition, the maximum deflection is typically less than 500 microns under a standard axle load. The pavement layers influence the deflection bow. The influence of the pavement structure on the deflection bow is illustrated by three different scenarios in the Figure 30:

- Scenario 1 is a stiff pavement, with a relatively stiff and strong cemented sub-base layer. The deflection is relatively low, and the bowl wide in comparison to its magnitude.
- Scenario 2 is a pavement that is relatively old, but has good quality materials. The deflection is higher than Scenario 1, because the pavement is less stiff.
- Scenario 3 is an old pavement with that has poor quality materials, and has a moist sub-base and sub-grade. The deflection is large, and the width of the bowl is narrow.



Scenario 1		Scenario 2		Scenario 3	
40 mm	Asphalt	60 mm	Old asphalt with seals	30 mm	Old asphalt
125 mm	G1 Base	250 mm	G2 Base	150 mm	G4 Base
250 mm	C3 Subbase	250 mm	G4 Subbase	250 mm	G5 Subbase
300 mm	G7 Selected	150 mm	G6 Selected	450 mm	G7 Selected
	G9 Subgrade		G9 Subgrade		G7 Subgrade

Figure 30 Example Deflection Bowls in a different pavement design

3.9. Pavement Structure using granular base (Inverted Pavement)

This type of pavement comprises a base of untreated gravel or crushed stone on a granular or cemented gravel sub-base with a sub-grade of various soils or gravels. The way of distressing, usually exhibited by these pavements, is largely controlled by the type of sub-base.

With a granular sub-base it is usually permanent (plastic) deformation arising from densification and/or shear of the untreated material. This deformation may manifest itself either as rutting or as surface roughness.

Cemented sub-base layers in granular base pavements generally increase the load carrying capacity of the pavement. Initially, cemented layers result in relatively high effective elastic modulus (stiffness) owing to cementation. Based on field measured deflections at various depths in these pavements, back-calculated linear elastic effective elastic modulus for pre-cracked cemented layers range between 1000 MPa and 1500 MPa (De Beer, 1985) (Freeme, Maree, & Viljoen, 1982). The failure way of a cemented sub-base is usually fatigue cracking and this results in a reduction of the effective elastic modulus as the layer progresses into the post-cracked state.

In this case an effective elastic modulus as low as 300 to 500 MPa can occur if the initial pavement structure is not well balanced and/or the original material is substandard. It is important to note that initial cracking, which causes a reduction in effective modulus, can be brought about by construction traffic.

The fatigue cracking in the cemented sub-base may propagate until eventually the layer exhibits properties similar to those of a natural granular material, i.e., an equivalent granular state. It is unlikely that cracking will reflect to the surface and it is likely that there will be little rutting or longitudinal deformation until after the sub-base has cracked extensively. However, if the sub-base exhibits relatively large shrinkage or thermal cracks, these may reflect through the surface.

The modulus of the cemented sub-base will depend on the quality of the parent material originally stabilised, the cementing agent, the effectiveness of the mixing process, the density achieved and the degree of cracking.

- **Granular Layer (G1 to G10)**

These materials show stress-dependent behaviour and, under repeated stresses, deformation can occur through shear and/or densification.

A G1 is a dense-graded, not weathered, crushed stone material compacted to 86 to 88 % of apparent relative density.

A G2 material is crushed stone material compacted to 100-102 % mod. AASHTO density or 85 % of bulk relative density. G2 and G3 may be a blend of crushed stone and other fine aggregate used to adjust the grading. From G4 to G10 materials cover the range of relatively high quality gravels o relatively lower quality materials used in pavement layers. As you can see in the Catalogue Designs shows in the Table 10.

Table 10 Material Symbols and Abbreviated Specifications in the Catalogue Designs

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	G1	Graded crushed stone	Dense - graded unweathered crushed stone; Maximum size 37,5 mm; 86 - 88 % apparent relative density; Soil fines PI < 4
	G2	Graded crushed stone	Dense - graded crushed stone; Maximum size 37,5 mm;100 - 102 % Mod. AASHTO or 85 % bulk relative density; Soil fines PI < 6
	G3	Graded crushed stone	Dense - graded stone and soil binder; Maximum size 37,5 mm; 98 - 100 % Mod. AASHTO ; Soil fines PI < 6
	G4	Crushed or natural gravel	Minimum CBR = 80 % @ 98 % Mod. AASHTO; Maximum size 37,5 mm; 98 - 100 % Mod. AASHTO; PI < 6; Maximum Swell 0,2 % @ 100 % Mod. AASHTO. For calcrete PI ≤ 8
	G5	Natural gravel	Minimum CBR = 45 % @ 95 % Mod. AASHTO; Maximum size 63 mm or 2/3 of layer thickness; Density as per prescribed layer usage; PI < 10; Maximum swell 0,5 % @ 100 % Mod. AASHTO *
	G6	Natural gravel	Minimum CBR = 25 % @ 95 % Mod. AASHTO; Maximum size 63 mm or 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12; Maximum swell 1,0 % @ 100 % Mod. AASHTO *
	G7	Gravel / Soil	Minimum CBR = 15 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G8	Gravel / Soil	Minimum CBR = 10 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G9	Gravel / Soil	Minimum CBR = 7 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G10	Gravel / Soil	Minimum CBR = 3 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; or 90% Mod. AASHTO

- **Cemented Layer (C1 to C4)**

Cemented materials, reported in the Table 11, similar to concrete, are initially elastic and have limited tensile strength and usually crack under repeated flexure.

An unbound layer covering (overlay) the cemented layers can be used to prevent penetration of reflective shrinkage cracks from the cemented layers to the surface. A C2 material may be used when an erosion-resistant layer is required. Both C3 and C4, materials can be used in place of natural gravel layers in bases and sub-bases (De Beer, 1985). They can be either cement treated or lime treated, depending on the properties of the natural materials. However, the longer term durability and resistance to erosion of these materials should be carefully assessed.

Table 11 Material Symbols and Abbreviated Specifications for cement mix layer in the Catalogue Designs

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	C1	Cemented crushed stone or gravel	UCS**** : 6,0 to 12,0 MPa at 100 % Mod. AASHTO; Specification at least G2 before treatment; Dense - graded ; Maximum aggregate 37,5 mm
	C2	Cemented crushed stone or gravel	UCS : 3,5 to 6,0 MPa at 100 % Mod. AASHTO; Minimum ITS ***** = 400 kPa at 95 - 97 % Mod. AASHTO compaction; Specification at least G2 or G4 before treatment; Dense - graded; Max. aggregate 37,5 mm; Max. fines loss = 5 %******
	C3	Cemented natural gravel	UCS : 1,5 to 3,5 MPa at 100 % Mod. AASHTO; Minimum ITS***** = 250 kPa at 95 - 97 % Mod. AASHTO compaction; Maximum aggregate 63 mm; 5 % Maximum PI = 6 after stabilization; Max. fines loss = 20 %
	C4	Cemented natural gravel	UCS : 0,75 to 1,5 MPa at 100 % Mod. AASHTO; Minimum ITS***** = 200 kPa at 95 - 97 % Mod. AASHTO compaction; Maximum aggregate 63 mm; 5 % Maximum PI = 6 after stabilization; Max. fines loss = 30 %

- **Surfacing Layer (AG to AP; S1 to S8)**

The surfacing covers, reported in the Table 12, the range from high-quality asphalt surfacing to surface treatments and surface maintenance measures such as rejuvenators and diluted emulsion treatmen**ts** (TRH8, 1987). They also include porous asphalt surfacing layers (AP) (SABITA, 1994)**.**

Table 12 Material Symbols and Abbreviated Specifications for surface layer in the Catalogue Designs

	AG	Asphalt surfacing	Gap graded (TRH 8, 1987)
	AC	Asphalt surfacing	Continuously graded (TRH 8, 1987)
	AS	Asphalt surfacing	Semi - gap graded (TRH 8, 1987)
	AO	Asphalt surfacing	Open graded (TRH 8, 1987)
	AP	Asphalt surfacing	Porous (Drainage) asphalt (SABITA, manual 17, 1994)
	S1	Surface treatment	Single seal (TRH 3, 1996)
	S2	Surface treatment	Multiple seal (TRH 3, 1996)
	S3	Sand seal	See TRH 3, 1996
	S4	Cape seal	See TRH 3, 1996
	S5	Slurry	Fine grading
	S6	Slurry	Medium grading
	S7	Slurry	Coarse grading
	S8	Surface renewal	Rejuvenator
	S9	Surface renewal	Diluted emulsion

Table 13 South African Road Catalogue for Dry Regions

GRANULAR BASES (DRY REGIONS) DATE 1996

ROAD CAT.	ES1 < 3000	ES2 0,3-1,0x10^{4}	ES3 1,0-3,0x10^{4}	ES4 3,0-10x10^{4}	ES5 0,1-0,3x10^{6}	ES6 0,3-1,0x10^{6}	ES7 1,0-3,0x10^{6}	ES8 3,0-10x10^{6}	ES9 10-30x10^{6}	ES10 30-100x10^{6}	Foundation
	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										
A							40A 125 G2 150 C3 40A 150 G2 150 G5	40A 150 G2 250 C3	50A . 150 G1 250 C3	50A 150 G1 300 C3	
B						S 125 G4 150 C4 S 150 G4 150 G5	S*/30A 150 G3 150 C4 S*/30A 150 G3 150 G5	40A 150 G2 200 C4 30A 150 G2 200 G5			150 G7 150 G9 G10
C				S 100 G5 125 C4 S 125 G4 125 G6	S 125 G5 125 C4 S 125 G4 150 G6	S 125 G4 125 C4 S 125 G4 150 G5	S 150 G3 150 C4 S 150 G3 150 G5				
D	S1 100 G5 100 G7	S1 100 G5 125 G7	S1 100 G4 125 G7	S1 100 G4 125 G6 S1 100 G5 100 C4	S 125 G4 125 G6 S 100 G5 125 C4	S 125 G4 150 G6 S 125 G5 150 C4					150 G9 G10

Table 14 South African Road Catalogue for Wet Regions

GRANULAR BASES (WET REGIONS) DATE 1996

ROAD CAT.	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										Foundation
	ES1 < 3000	ES2 $0,3\text{-}1,0\times10^4$	ES3 $1,0\text{-}3,0\times10^4$	ES4 $3,0\text{-}10\times10^4$	ES5 $0,1\text{-}0,3\times10^6$	ES6 $0,3\text{-}1,0\times10^6$	ES7 $1,0\text{-}3,0\times10^6$	ES8 $3,0\text{-}10\times10^6$	ES9 $10\text{-}30\times10^6$	ES10 $30\text{-}100\times10^6$	
A							30A 150 G1** 200 C3	40A 150 G1 300 C3 *(250 C3)	50A 150 G1 400 C3 *(300 C3)		
B						S 150 G2 150 C4 S 150 G2 200 G5	S/30A 150 G1** 200 C4	40A 150 G1 300 C4 *(250 C4)			150 G7 150 G9 G10
C				S 100 G5 125 C4 S 125 G4 125 G6	S 125 G5 125 C4 S 150 G4 150 G6	S 125 G2 150 C4 S 150 G2 150 G5	S 150 G2** 200 C4 S 150 G2 150 G4				
D	S1 100 G5 100 G7	S1 100 G5 125 G7	S1 100 G4 125 G7	S1 100 G4 125 G6 S1 100 G5 100 C4	S 125 G4 125 G6 S 100 G5 125 C4	S 150 G4 150 G6 S 125 G5 150 C4					150 G9 G10

4. INVERTED BASE PAVEMENTS: STATE OF ART

4.1. Introduction

On the base of South African experience other countries have started studying these kind of paving. During the 1980s, France developed other types of pavement structures trying to limit the use of asphalt materials because of the high cost of bitumen. At this time, the French design guide recommends inverted base pavements to prevent reflective cracking between cohesive layers (Corté & and Goux, 1996)**.**

The study concluded in USA shows that the inverted base pavement perform better than the conventional design in terms of cracking density and severity after 10 years, and it also offered a superior ride quality over the evaluation period (Cortes A. D., 2010) (Rasoulian, Titi, Martinez, Becnel, & Keel, 2001) (Barksdale R. D., 1984).

This chapter starts with a comprehensive review of inverted base pavement research in South Africa and USA, including past and recent case histories.

4.2. South Africa experience

South Africa developed the Inverted Pavement Technique and it keeps on studying this one according to the continuous growing axial loads that encumber on the infrastructure. The study of the flexible paving has been increasing a lot from 1978 to 1994 thanks to the introduction of the Accelerated Pavement Testing (APT) facilities such as HVS (Heavy Vehicle Simulator) that allowed to make pavement load simulations and therefore to improve some aspects. South Africa has reported a 20-25% cost savings compared to conventional portland cement concrete (PCC) or hot mix asphalt (HMA) pavements. (Lewis, Ledford, Georges, & Jared, 2012) Many South African studies have highlighted the importance of one IP layer known as G1.

The N1 highway, north of Pretoria is an example of efficiency of the South African highways, realized about 26 years ago using the G1 layer as a base layer, it is still used and in good condition. Notwithstanding the great valuable experience from the numerous roads built with this technique, additional test section have been built in recent years as the experimental section on the R104 between Pretoria and Bronkhorstspruit.

4.3. USA Experience

The United States transportation system is a vast network of roads, and despite the high expenses, the infrastructure remains in bad condition. For this reason there are more studies and case history, resume in Table 15, about new alternatives with lower life-cycle costs, to investigate the potential of inverted base pavements as a reliable and more economical alternative to conventional pavements (Papadopoulos, 2014).

4.4. New Mexico (1954)

The first application of inverted pavements in the United States can be traced back to 1954 in New Mexico (Tutumluer E. , 2013)**.**

Johnson studied this experimental project with particular interest at a possible degradation of the mineral aggregates. He reported that after six years of heavy traffic, no reflection cracking or significant rutting had developed in the test sections (Johnson, 1961)

Subsequently, following these early successful uses of " inverted sections " or "upside down", experimental roads were constructed in New Mexico on U.S. 64 North of Santa Fe consisted of a 3 in. (76 mm) asphalt concrete surfacing, 6 in. (152 mm) granular base and a 6 in. (152 mm) granular sub-base treated with 4% cement and another on the Roads Forks-East project in New Mexico. (Barksdale & Todres, 1983)

4.5. U.S. Corps of Engineers (1970's)

The US Army Corps of Engineers tested two almost inverted base pavements, both composed of a 0.09m asphalt concrete layer, a 0.15m crushed limestone base, a 0.38m stabilized clay sub-base, and a clay sub-grade (CBR= 4). The structures were subjected to traffic under controlled conditions and noticed the displacements and stresses at key locations (Ahlvin, Turnbull, Sale, & and Maxwell, 1971) (Barker, Brabston, & and Townsend, 1973) (Grau, 1973).

4.6. Georgia Tech (1980's)

In the 1980s, at Georgia Tech two inverted base designs were tested as part of an extensive laboratory study (Barksdale R. D., 1984)**.** The two inverted pavements structure consisted of a 0.09m asphalt concrete layer, a 0.20m unbound aggregate layer (well graded granitic gneiss), and a 0.15m cement-treated base over a micaceous silty sand sub-grade. The asphalt surface layer was a GDOT-B binder mixed with granitic gneiss, laid in 0.04m lifts. It was found that the cement-treated base facilitates compaction in inverted structures leading to denser unbound aggregate layers (Barksdale 1984). It was found that the two inverted base pavement sections outperformed equivalent pavement structures in terms of lower resilient surface displacements, reduced transferred compressive-stress onto the sub-grade and less tensile-radial-strain at the bottom of the asphalt concrete layer. The superior mechanical performance of the inverted pavement structures was also reflected in the number of load cycles to failure (Cortes A. D., 2010)**.**

4.7. Route LA97 and Accelerated Pavement Testing - Louisiana (1990's)

The inverted pavements structure that was tested in Louisiana (1991-2001), consisted of a 0.09m asphalt concrete layer, a 0.10m crushed limestone base, and a 0.15m cement-treated base. The measurement under both field and accelerated pavement testing conditions in an effort to reduce reflective cracking occurring in full depth soil-cement pavements (Metcalf, Romanoschi, Li, & Rasoulian, 1999) (Titi, Rasoulian, Martinez, Becnel, & Keel, 2003)**.** The inverted pavement structure carried 4.7 times more ESAL's than the conventional flexible pavement on a cement-stabilized-base. (Rasoulian, Becnel, & Keel, Stone interlayer pavement design. , 2000)

4.8. Morgan County Quarry Access Road – Georgia (early 2000's)

The Morgan County inverted pavement test sections consist of a 0.08m asphalt concrete layer, a 0.15m crushed Georgia granite base, and a 0.20m cement-treated base. The aim of this project was to compare the effectiveness of South African and Georgian compaction practices. The test sections have experienced uninterrupted high-volume heavy-truck traffic for 9 years. Surveys conducted in May 2008 after the section had serviced over 1.2 million ESALs in 7 years (75% of the designed service life), found no signs of distress or changes in ride quality (Lewis D. E., 2009) (Cortes A. D., 2010)

4.9. LaGrange Georgia (2010's)

The test section build in LaGrange was starting in 2009, is part of an industrial parkway intended to serve the growing car manufacturing industry in south-west Georgia. The pavement is a two-lane 1036m long and it was designed to sustain an initial one-way annual average traffic of 7000 vehicles per day projected to grow to 11700 by the end of its service life.

The structure consists of a 0.09m asphalt concrete layer, a 0.15m crushed Georgia granite base, and a 0.20m cement-treated base and it was designed by using empirical guidelines from the South African experience. (Cortes A. D., 2010) (Terrell, Cox, Stokoe, Allen, & Lewis, 2003) The pavement was opened to traffic in April 2009 and the first performance data was expected in 2011. The documented construction of the Lagrange project shows that no special equipment is required for construction of inverted base pavements (Papadopoulos, 2014)**.** The comparison among the life-cycle costs for the LaGrange inverted pavement sections and a rigid pavement designed to carry the same amount of traffic is presented by (Buchanan, 2010) where the inverted pavement section results in net savings of $139,000 over a 30-year period.

4.10. Luck Stone Bull Run Project - Virginia (2010's)

An application in 2010 of inverted pavement in Virginia involved a relocated road (Virginia Highway 659) bypassing the Luck Stone Bull Run Quarr**y.** (Tutumluer E. , 2013).The possible different approach (Anisotropic and Isotropic) to predicted rutting life, the pavements response analysis in terms of vertical and horizontal stress is presented by (Weingart, 2012), as well as the benefits of this inverted pavement trial application. He reported a potential for 22.3% cost savings compared to the construction of a conventional flexible pavement with equivalent structural and functional capacities; the estimated cost for a construction of the conventional flexible pavement section was $21,311 per 100 linear ft, whereas the one for the inverted pavement section was $16,555 per 100 linear ft.

Table 15 Inverted base pavement case histories in the US

Location	**Year**	**Layer thickness from top to bottom (mm)**		**Reference**
I-010-1 Road Forks - East Mexico	1969	AC (38) UAB (152) CTB (152)	AC (76) UAB (152) CTB (152)	Johnson, 1961
F-51-1 Santa Fe New Mexico	1960	AC (76) UAB (152) CTB (152)		Johnson, 1961
US Army Corps Viclsburg, MS	1971	AC (76) UAB(152) Stabilized Clay Subbase(381)		Ahlvin et al. 1971 Barker et al. 1973
Gerogia Tech Atlanta, GA	1980	AC (89) UAB (203) CTB (152)		Barksdale 1984 Barksdale et al. 1983
Louisiana	1991	AC (89) UAB (102) Soil Cement (152)		Metealf et al. 1999 Raouslian et al. 2000 Titi et al. 2003
Morgan County,	1999	AC (76)		Lewis et al. 2012

GA		UAB (152) CTB (203) Filler (51) Prepared Subgrade (CBR 15)	Terrel et al. 2003
Lagrange, GA	2008	AC (89) UAB (152) CTB (254) Stabilized SubGrade (152)	Cortes, 2010
Bull Run, VA	2010	AC (127) UAB (152) CTB (254) Prepared Subgrade	Weingart, 2010

Note: AC: asphalt concrete, UAB: unbound aggregate base, CTB: cement treated base

5. BEHAVIOR OF UNBOUND AGGREGATE BASE

5.1. Introduction

Lots of researchers have focused the attention on the anisotropic property of granular bases, i.e., the directional dependency of material properties. (Adu-Osei, Little, & Lytton, 2001) (Rowshanzamir, 1997) (Tutumluer & R., Anisotropic modeling of granular bases in flexible pavements, 1997). Mechanical behavior of particulate materials is significantly influenced by the directional dependency of material properties. This behavior can be accommodated using the anisotropic characterization of geo materials used in pavement. The isotropic characterization of layered systems typically results in high tensile stresses in the intermediate layers when the stiffness of subsequent layers are significantly different (Reza Salehi & Little, 2009). Tutumluer showed that stresses and strains measured in the field had a much closer match to calculated responses when the aggregate layers were characterized as anisotropic materials (Tutumluer, Little, & Kim, 2003).

5.2. Anisotropic Behavior

The anisotropic behavior of aggregate systems can be investigated within the framework of elasticity. The general form of constitutive behavior is the generalized Hooke's law. In generalized Hooke's law, any strain component can be found as a function of all the stress components that act on the body of a material.

$$\varepsilon_{ij} = S_{ijkl}\,\sigma_{kl}$$

where: ε represent the strain; σ is the stress; and S is known as the stiffness or compliance matrix that relates the applied stresses and measured strains. In the general form, the compliance matrix has (3^4=81) components. Symmetry of the response of the geo materials results in reduction of the number of components in the compliance matrix from 81 to 36.

$$\begin{bmatrix} S_{12} & S_{12} & S_{13} & S_{14} & S_{15} & S_{16} \\ S_{21} & S_{22} & S_{23} & S_{24} & S_{25} & S_{26} \\ S_{31} & S_{32} & S_{33} & S_{34} & S_{35} & S_{36} \\ S_{41} & S_{42} & S_{43} & S_{44} & S_{45} & S_{46} \\ S_{51} & S_{52} & S_{53} & S_{54} & S_{55} & S_{56} \\ S_{61} & S_{62} & S_{63} & S_{64} & S_{65} & S_{66} \end{bmatrix} \begin{bmatrix} \sigma_x \\ \sigma_y \\ \sigma_x \\ \tau_{yz} \\ \tau_{zx} \\ \tau_{xy} \end{bmatrix} = \begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \gamma_{yz} \\ \gamma_{zx} \\ \gamma_{xy} \end{bmatrix} \qquad (2.2)$$

where ε and γ represent the strain components, and σ and τ are the stress components in the constitutive equation.

Love showed for an elastic material the compliance matrix (S matrix) should be symmetrical because of thermodynamic requirements and strain energy considerations (Love, 1994). Therefore:

$$S_{ij}=S_{ji}$$

This consideration reduce the number of the elastic constants in the compliance matrix from 36 to 21.

Therefore, for a general anisotropic material, we need to define 21 components in the compliance matrix to fully characterize the anisotropic behavior of the geo materials.

However, aggregate systems generally show symmetry in terms of response behavior under the load between the normal and shear component of the stress and strains (Lekhnitskii, 1963).

Hoek showed that the presence of the axis of symmetry in the material de-couples some of the stress-strains relations and reduces the number of independent material constants required to characterize anisotropic behavior of a material (Hoek & Brown, 1997). For orthotropic materials in which the materials axis of symmetry coincides with loading directions, the number of independent components can be reduced to 9.

Material behavior	Number of distinct material constants
General anisotropic material	81
Anisotropic material considering symmetry of stress strain tensor ($\sigma_{ij}=\sigma_{ji}$, $\varepsilon_{ij}=\varepsilon_{ji}$)	36
Anisotropic material considering elastic energy considerations	21
General orthotropic material	9
Orthotropic material with transverse isotropy (cross-anisotropic material)	5
Isotropic material	2

During specimen preparation and compaction elongated aggregate particles tend to align with the large dimension parallel to the horizontal direction (Cortes & Santamarina, 2013) (Reza Salehi & Little, 2009). This preferred the orientation of the aggregate particles results in directional dependency of the material properties in aggregate layers. One simplifying assumption in modeling, represented in the Figure 31, the aggregate layers would be to consider the aggregate system is cross anisotropic. In other words, the material is considered to be isotropic in the horizontal plane and anisotropic in vertical plane. For this reason it is also known as transversely isotropic material. In transversely isotropic materials, mechanical properties in horizontal plane are the same, and they change with depth in the aggregate layer. Ishai defined the cross anisotropic geo materials as "an orthotropic material is called transversely isotropic (or cross- anisotropic) when one of the principal planes is the plane of isotropy, that is, at every point there is a plane on which the mechanical properties are the same in all directions (Ishai, 1994).

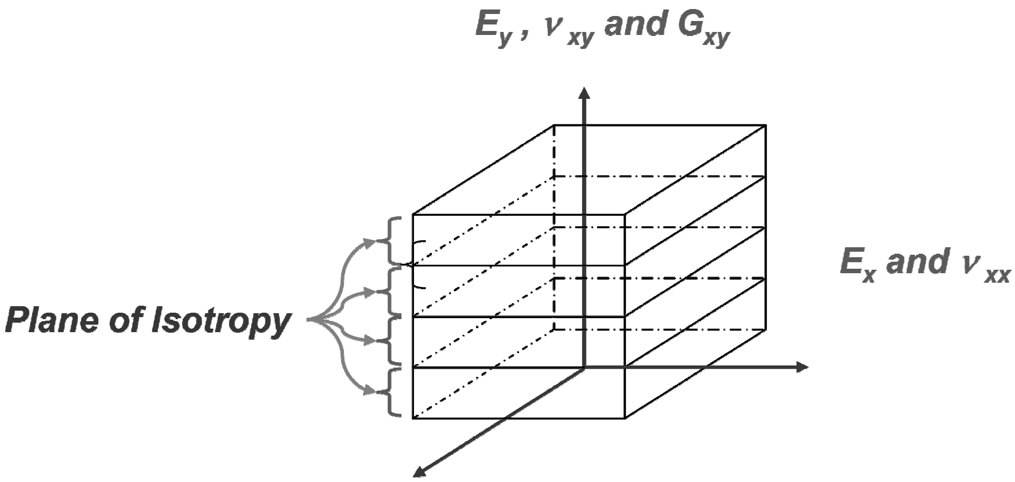


Figure 31 Schematic representation of cross-anisotropic materials

5.3. Cross-Anisotropic material

As specified above, granular material layer can be studied as a cross-anisotropic material. The stress strain relations presented in equation (2-2) can be written as:

$$\sigma_{11} = S_{11}\,\varepsilon_{11} + S_{12}\,\varepsilon_{22} + S_{12}\,\varepsilon_{33} \qquad (2\text{-}4)$$

$$\sigma_{22} = S_{12}\,\varepsilon_{11} + S_{22}\,\varepsilon_{22} + S_{22}\,\varepsilon_{33} \qquad (2\text{-}5)$$

$$\sigma_{22} = S_{12}\,\varepsilon_{11} + S_{22}\,\varepsilon_{22} + S_{22}\,\varepsilon_{33} \qquad (2\text{-}6)$$

$$\sigma_{12} = 2S_{66}\,\varepsilon_{12} \qquad (2\text{-}7)$$

$$\sigma_{31} = 2\,S_{66}\,\varepsilon_{31} \qquad (2\text{-}8)$$

$$\sigma_{23} = (S_{22} - S_{23})\varepsilon_{12} \tag{2-9}$$

From this we can understand that the study of cross-anisotropic material is subject to the characterization of five constants. These material constants acquire more physical meaning when they are expressed using engineering constants such as modulus and Poisson's ratios. These engineering constants can be experimentally found in the lab using triaxial cell or hollow cylinder for geo materials. The constitutive relation presented in equation 2-2 can be expressed in terms of engineering constants while considering the symmetry of the material as follows:

$$\begin{bmatrix} \frac{1}{E_x} & \frac{-\upsilon_{yx}}{E_y} & \frac{-\upsilon_{yx}}{E_y} & 0 & 0 & 0 \\ \frac{-\upsilon_{xy}}{E_x} & \frac{1}{E_y} & \frac{-\upsilon_{zy}}{E_y} & 0 & 0 & 0 \\ \frac{-\upsilon_{xy}}{E_x} & \frac{-\upsilon_{yz}}{E_y} & \frac{1}{E_y} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{yz}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{xz}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{xy}} \end{bmatrix} \begin{bmatrix} \sigma_x \\ \sigma_y \\ \sigma_x \\ \tau_{yz} \\ \tau_{zx} \\ \tau_{xy} \end{bmatrix} = \begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \gamma_{yz} \\ \gamma_{zx} \\ \gamma_{xy} \end{bmatrix} \tag{2-10}$$

E_x= Elastic modulus in horizontal direction

E_y= Elastic modulus in vertical direction

ν_{yx}= Poisson's ratio in horizontal direction due to imposed vertical stress

ν_{xy}= Poisson's ratio in vertical direction due to imposed horizontal stress

ν_{xy}= Poisson's ratio in horizontal direction due to imposed horizontal stress

G_{xz}= Shear modulus in x-z plane

G_{yz}= Shear modulus in y-z plane

G_{xy}= Shear modulus in x-y plane

It is important to note that using ($\nu_{xy} \neq \nu_{yx}$) from equation 2-10 and the symmetry of the compliance matrix we have:

$$\frac{\upsilon_{xy}}{E_x} = \frac{\upsilon_{yx}}{E_y} \tag{2-11}$$

Since the horizontal plane is the plane of isotropy, shear modulus in horizontal direction (G_{xx}) is related to elastic modulus and Poisson's ratio in horizontal plane following equation 2-13:

$$G_{xx} = \frac{E_x}{2\,(1+\nu_{xx})} \tag{2-13}$$

Considering equations 2-11 and 2-13, the number of independent elastic constants can be reduced to five.

$$\begin{bmatrix} \frac{1}{E_x} & \frac{-\upsilon_{xx}}{E_x} & \frac{-\upsilon_{yx}}{E_y} & 0 & 0 & 0 \\ \frac{-\upsilon_{xx}}{E_x} & \frac{1}{E_x} & \frac{-\upsilon_{yx}}{E_y} & 0 & 0 & 0 \\ \frac{-\upsilon_{yx}}{E_y} & \frac{-\upsilon_{yx}}{E_y} & \frac{1}{E_y} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{xy}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{xy}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{2(1+\upsilon_{xx})}{E_x} \end{bmatrix} \begin{bmatrix} \sigma_x \\ \sigma_y \\ \sigma_x \\ \tau_{yz} \\ \tau_{zx} \\ \tau_{xy} \end{bmatrix} = \begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \gamma_{yz} \\ \gamma_{zx} \\ \gamma_{xy} \end{bmatrix} \tag{2-14}$$

Equation 2-14 indicates that five anisotropic elastic material properties: E_x, E_y, ν_{xy}, ν_{xx}, and G_{xy} are needed to fully characterize the cross-anisotropic nature of geo materials.

5.4. Anisotropy in Unbound Aggregate Base

Material properties in different directions vary for deposited materials as well as compacted aggregate layers. The unbound granular layers exhibit stress- and direction-dependent (anisotropic) behaviour attributable to the nature of the granular medium and the orientation of aggregate. (Uzan, 1992). Directional dependency of material properties, significantly influences the distribution of stresses and strains in the aggregate systems.

In most engineering materials we can find or assume a single symmetry plane, three planes of symmetry as in orthotropic materials, or a single axis of symmetry. Most geological materials fall into the last category. Materials with one axis of symmetry have similar material properties along that axis. Such materials are defined as transversely isotropic or cross-anisotropic materials (Reza Salehi & Little, 2009). The anisotropy can be studied in two categories:

- Inherent anisotropy
- Stress-Induced anisotropy

5.5. Inherent anisotropy

Inherent anisotropy is considered as a physical characteristic inherent in the material and entirely independent of the applied strain (Arthur & Menzies, 1972). This intrinsic property of the naturally found geo materials, mainly built in geo materials structure from the influence of gravity and particle shape orientation during the deposit process, describes the structure of undisturbed soil deposit. (Umit, 2002). Size, shape, angularity, and texture of the aggregate particles have a significantly influence to the anisotropic behavior of granular material. The primary reason for this behavior is the fact that random assemblies have random distribution of inter-particle contacts. Due to the random nature of the orientation particles distribution and consequently random distribution of particle contacts, particulate systems exhibit directional dependency even though they are subjected to hydrostatic stresses. This type of anisotropy is called Inherent anisotropy.

5.6. Stress- Induced Anisotropy

Stress-induced anisotropy is defined as a physical property of a soil due only to strain associated with an applied stress. An isotropic material will behave isotropically under the application of an isotropic loading. However, when the applied stress is no longer isotropic, in other words it is not the same in all directions, the material will no longer strain isotropically. Therefore, new contact points occur and the stress/strain behaviors vertically and horizontally will no longer be in the same relationship to each other as before. (Karasahin & Dawson, 1993)

By definition induced anisotropy is an essential part of the straining process of a soil, but it is difficult to study comprehensively, because a vital feature of such a study is the controlled rotation of principal stress directions during shear. (Arthur, Chua, & Dunstan, Induced anisotropy in a sand, 1977)

6. PROPERTIES OF ROAD CONSTRUCTION MATERIAL

In this chapter it is represented a panoramic of the type of rocks used in South Africa to build infrastructures. The main mechanical characteristics are especially described by analyzing the issues linked to the type of minerals that are in the rocks and the way they affect the resistance and durability of one layer of the paving if altered. South Africa has an important experience on the making of IP (Inverted pavement) and on the determining the eligibility of the used materials. This will allow a comparison with the materials extracted in Sardinia and, furthermore, it will help having guide lines to understand which materials could be used for the realization of an IP.

6.1. South Africa classification of rock types for road construction purpose

Many types of rock are used in road construction in southern Africa, each rock type is distinguished from other by structural, textural and mineralogical characteristics and each one should be treated on its own merits. This is of course impractical, the South Africa approach consists in to group these rocks on the basis of similar road construction properties (TRH 14, 1985). This grouping is based on the presence or absence of quartz mineral and the effect that has on the weathering of the rock. Since *quartz* is so important for the assessment of the durability of a natural road building material, every road engineer should be able to recognize this mineral. The SA (South Africa) specification say that since every rocks is suitable for use somewhere in a pavement, the rock can only be rated as "good" or "bad" in terms of the various classes of road and their structural layers. The rock group are listed as follows:

-*Basic crystalline rocks:* Dolerite (andesite, basalt)

-*Acid crystalline rocks:* Granite (granite, gneiss)

-*High silica rocks:* quarzite (quarzite)

-*Arenaceous rocks:* sandstone

-*Argillaceousrocks:* shale

-*Carbonate rocks:* dolomite

-*Diamictites:* tillite

-*Metalliferous rocks:* ironestone

-*Pedogenic materials:* calcrete and ferricrete

Acid and basic crystalline rock are the most frequently used natural materials in more than half of the surface area of South Africa.

6.2. Crushing Strength

As suggested by South Africa specification, strength is one of the most important criteria for the assessment of durability and quality. In case of road construction, strength or crushing strength is defined as a natural property of rock which varies with the type of rock and its stage of weathering. For crushing road aggregate can be considered only fresh or slightly weathered rock, the crushing strength is not only a measure of a suitability of the rock, but it is also a measure of the degree of disintegration which the rock has attained and thus its durability. The crushing strength value is obtained from the 10 per cent Fines Aggregate Crushing Test (Table 16), and Aggregate Crushing Value test (Table 17) . The aggregate used for crushed stone base shall comply with the requirements specified in the table. It shall not contain any deleterious materials such as weathered rocks, clay, shale or mica. (COLTO, 1998)

Table 16 10% Fines Aggregate Crushing Values (10%FACT)

Rock type	Matrix	Dry min	Wet min	Wt/Dry relationship
Arenaceous rocks	Non-siliceous cementing material	140 kN	-	75%
	siliceous cementing material	110 kN	-	75%
Diamictites (tillite)		200 kN	-	70%
Argillaceous rocks		180 kN	125 kN	-
Other rock types		110 kN	-	75%

Table 17 Aggregate crushing value (ACV)

Rock type	ACV, max
Arenaceous: without siliceous cementing matrix	27%
Arenaceous: with siliceous cementing matrix	29%
Diamictites (tillite)	21%
Argillaceous rocks	24%
Other rock types	29%

6.3. Influence of weathering on road aggregates

When natural aggregate other than fresh rock are selected, the environmental influence must not be disregarded, since it determines the mode and the stage of weathering of the material and its possible durability. The environmental factors considered in the South African specification are climate and topography. Climate is the main factor that influences the weathering and the durability of natural aggregate. The climate conditions are expressed by the N-value:

$$N = \frac{12\,Ej}{Pa}$$

where, *Ej* is the computed evaporation during January and *Pa* is the total annual precipitation. There are only three N-values of importance to road construction and they are N=2, N=5 and N=10.

The most important is N=5. When N is less than 5, many rocks, especially the crystalline types, decompose; When N is higher than 5, all rock disintegrate. Decomposition and disintegration are the main forms of weathering and decomposition is more detrimental to the quality and durability of natural aggregate than disintegration.

In *decomposition* secondary minerals consisting mostly of a clay are formed and an entire different material develops. This process affects only those minerals which are known as "primary" and it is restricted to the acid and basic crystalline rock. The state of decomposition must be considered in relation to the N-value at the site because the rate of decomposition increases within a pavement layer. This is best done by determining the percentage of a secondary minerals. The maximum percentage of secondary minerals permissible to ensure adequate durability of a crystalline rock when used in bases or sub-base is shown in the Figure 32

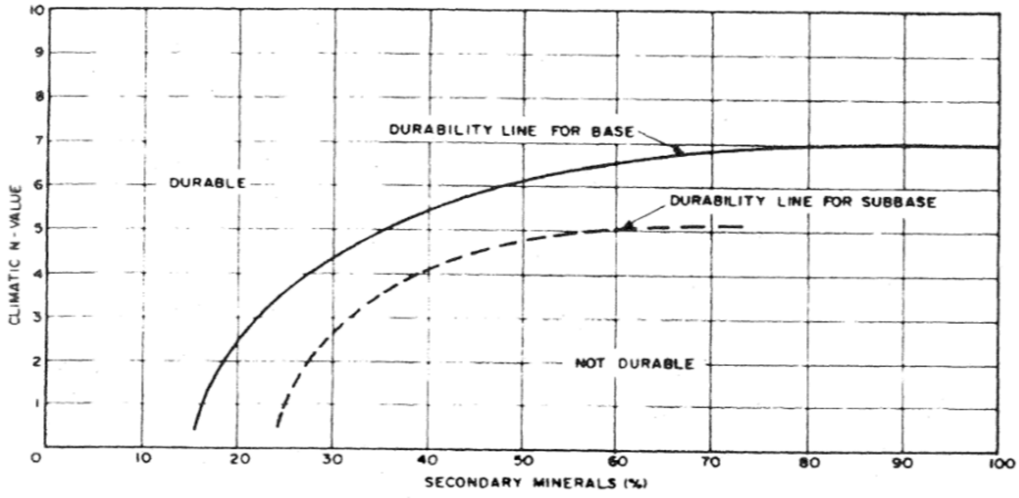


Figure 32 Durability of weathered crystalline rocks in pavements depending on the N-Value and the percentage of secondary minerals

In *disintegration* the rock merely breaks and crumbles down to even smaller pieces while its mineral remain unaffected. Acid and basic crystalline rock and certain diamictites disintegrate only where N is higher than 5, but all types of rock disintegrate in all climatic environmental. The effect of this weathering on aggregate property is a continuous decrease in strength without the durability of the material begin affected. This means that any material which satisfies the design requirement can be used without the risk that it will change in service to the extent that it could cause problem.

6.4. Polishing

The resistance to abrasion is a natural property of rock which depends mainly on their mineral composition, but textural features and weathering may have a modifying effect. The effect of mineral to abrasion is largely function of their hardness. The hardness of mineral that forming the rock according to Mohs' scale is presented in the Table 18

Table 18 Hardness of rock-forming minerals (Mohs' scale)

Mineral	Hardness
Quartz	7
Olivine	6,5 - 7
Pyroxenes	6 - 6,5
Feldspars	6
Opal	5,5 - 6,5
Amphibole	5,5 - 6
Dolomite	3,5
Calcite	3
Micas	2,5 - 3
Chlorite	2 - 2,5
Clay mineral	1 - 2

From this table it is possible to notice that *quartz* is the hardest mineral of rock and the *olivina* is almost as hard. It is obvious that a rock composed by mineral of different hardness will polish less than a rock whose mineral are all about the same hardness. The difference in hardness of the components is more important than their absolute hardness, and therefore rock in compose of quartz only will also polish eventually.

The distribution of minerals in a rock gives it a textural property such that the more evenly hard and soft minerals distributed within a rock, the less the rock will polish. Thus the occurrence of olivine in certain Basalts (olivine basalt) is of little advantage to the polishing resistance of the rock because olivine occur mostly in clusters. If such basalt is crushed, many chip will be entirely free of olivine, and where a cluster of olivine occurs in a chip, the olivine will be inclined to break out, leaving an olivine-free chip behind. Very slight weathering, especially decomposition of crystalline rock, for example if the percentage of secondary minerals is between 10 an 15, may cause the polishing to be less severe, especially in quartz-free rock. At this stage of weathering, only a limited number of minerals are affected by decomposition, and even if the rock were composed only of a mineral of similar hardness, the slightly weathered minerals would abrade faster than not weathered ones, and the rough surface texture of the chip would be retained. (TRH 14, 1985).

The Basic crystalline rock are liable to polish due either to some distributed olivine (dolerite) or to slight weathering. Generally Acid crystalline rock do not polish much, whereas High silica rock polish in time but their great overall resistance makes the binder abrade faster than the aggregate and a rough surface is retained

6.5. Flakiness phenomenon

The flakiness is a natural property of rock, defined like the tendency of a rock to break into cubical or flaky chips during crushing. *Shistose,* laminated, very fine grained or dense rock are more inclined to produce flaky chips than are rock of another texture. However, the technique of crushing, especially the rate of reduction, probably has a greater influence on the production of cubical or flaky chips than the petrological nature of rock. Cubical chips could thus the produced from any type of rock and would be durable if produced from fine to medium grained or dense rock. However, cubical chips from shistose or laminated rock will not last because they will crack in change into little plates during handling and in service. This cracking will occur along the predetermined plans of weakness, i.e. the layer of shistosy or lamination. It may often be difficult to crush coarse grained rock satisfactorily because the crushing process may enhance the breakdown of the rock into its individual minerals. Cubical chips obtained from such rock may also be insufficiently durable.

The production of more or less flaky chips is mainly due to the crushing process, an assessment of the relationship between the groups of natural rock and the greater or lesser tendency of rock to break into flaky chips would not be very revealing. In general, it can only be stated that very fine grained and dense rock have a greater tendency to break into flaky chips than others, a fact which was known to prehistoric man who selected such rock for making his stone implements.

6.6. Deleterious minerals

A number of mineral which may occurs in rock from various group of rock may affect the performance property of pavement structure. The mineral and their characteristics are briefly describe follow:

6.6.1. Mica

Mica is a very elastic mineral, especially the light-colored muscovite, the plates of which, if bent tend to return to their original shape This spring action leads to an increase in the voids ratio with undesirable consequences for compacted layer and concrete. Compacted layers lose their density and the increased voids ratio promotes the ingress of water. Muscovite in concrete aggregate increases the water demand and drying shrinkage, decreases the cube strength and has an adverse effect on the workability. These problem occur particularly if the muscovite platelets have a diameter of more than 0,5 mm and if quantity is near to, or more than, 10 per cent by volume of the total aggregate. Cement stabilization helps to counteract the "spring action" to some degree.

Since the determination of the mica content is complex, if it can be recognized at a glance, i.e. without searching for the platelets, perhaps whit a hand lens, caution is required.

6.6.2. Clay minerals

These are part of a large group of rather different minerals. They all form platy crystal based on silica sheets and all have the ability to absorb water in their crystal structure or to release it if condition changes. The two most important groups of clay mineral, in pavements structure, are the *Kaolinite* and the *Smectite* groups. The best known member of the latter group is montmorillonite.

6.6.3. Sulphide minerals

These minerals, especially Pyrite (colour gold), Marcasite (color silver) and Chalcopyrite, also copper Pyrite (color iridescent gold, red and green) decompose easily in the presence of air and water. The decomposition is aggravated by the presence of certain bacteria which occur frequently in nature. One of result of decomposition of sulphide minerals is the formation of acids, especially sulphuric acid, with deleterious effect on concrate and lime or cement stabilized road layers. These acid are not stable and change into sulphate salts.

Experience has shown that the total quantity of sulphide minerals as well as their distribution in the rock determines the severity of their deleterious effect on roads layers.

An aggregate contains more than 1 per cent by volume of such minerals, particularly if they are evenly distributed through the rock and not concentrated in clusters here and there, should not be used in the base and sub-base.

6.6.4. Soluble salts

These do not only develop from the decomposition of sulphide minerals but may also be contained in the natural soil. The easiest method for the determination of potentially harmful quantities of soluble salts in road building materials is to measure the *electrical conductivity* of the soil paste. Depending on the grain size of the tested aggregate the South Africa specification suggest the following limits:

Size -0,425 mm: not more than 2,0 mS/cm at 25°

Size -6,7 mm : not more than 1,5 mS/cm at 25°

Material which give higher conductivity value should be rejected.

Notwithstanding the above, if the conductivity is :

- more than 0,04 mS/cm on the -0,425 mm material, or
- more than 0,02 mS/cm on the -6,7 mm material,

The quantity of sulphate, expressed as SO_3, should always be determined. The maximum sulphate content, expressed as SO_3 should not exceed 0,25 per cent if the PI or the -0,002 mm fraction is greater than 12 %, but it may be up to 1% SO_3 if both the PI and the -0,002 mm fraction are less than 12% before stabilization and if the pH of the saturated soils fines paste is not less than 6. The determination of the pH is required because sulphate in the quantities given above always involves the risk of some free sulphuric acid being present.

6.6.5. Reactive silica

This occurs in nature in a variety of forms which may have an adverse effect on concrete made with certain cements, especially those known as "high alkali" cements. Such form of reactive silica are *amorphous silica*, the mineral *opal* and *siliceous skeletons* of microfossil. These three modifications are often called amorphous silica. There are also the mineral *cristoballite* and *tridymite*, high temperature modifications of quartz, which become unstable under surface conditions, and the mineral *chalcedorry*, a cryptocrystalline (extremely fine grained) fibrous form of quartz which develops over geological time from amorphous silica. Even *quartz* itself, when it is strained owing to metamorphism or is intensely fractured, may be unsuitable for use with certain cement if these minerals make up 30% or more of the constituents of the aggregate. The number of rock, which may contain one or more of these form of silica, is so large.

6.6.6. Nepheline/anailcime

Nepheline is a primary mineral in certain basic crystalline rock, and analcime, a zeolite type mineral, is an alteration product of nepheline. The change from nepheline to anailcime is associated with a considerable increase in the volume of the new mineral, which in turn is associated with the complete disintegration of rock. This change occur rapidly if the rock is exposed to the atmosphere. It is particularly severe if the rock contains cryptocrystalline or amorphous nepheline. The process has been known as "sunbrun" and occurs particularly in nepheline basalt, but it cannot be completely excluded from other Basalts and rock such as phonolite and Andesite.

A rock which is liable to sunbrun may look entirely fresh when first exposed. The sunbrun potential may be determined by boiling pieces of the suspect rock in distilled water for at least 36 hours. If the rock is a sunbruner, white spots will be seen on the treated surface. Sunbrun has not yet been proved in southern Africa although a few case of breakdown of surface aggregate may have been the results of this process.

7. SARDINIAN ROCK AND SELECTION PROCEDURE

7.1. Introduction

Sardinia, with its 24,089 km ² is the second largest Island in the Mediterranean after Sicily. In Sardinia sedimentary, magmatic and metamorphic records are well represented within three main lithological complexes Figure 33: (Rombi, 2013)

- Paleozoic basement that underwent repeated phases of deformation and metamorphism during the Caledonian and Hercynian orogenic cycles, and was eventually intruded extensively by calc-alkaline granitoids
- Late Palaeozoic epicontinental sequence and a Mesozoic carbonate platform sequence, representative of stable shelves, that formed the passive margin of Southern Europe
- Cenozoic to Quaternary cover consisting of shallow-water marine carbonates, siliciclastic sediments, continental conglomerates, as well as volcanic rocks represented by a calc-alkaline suite and alkaline basalts.

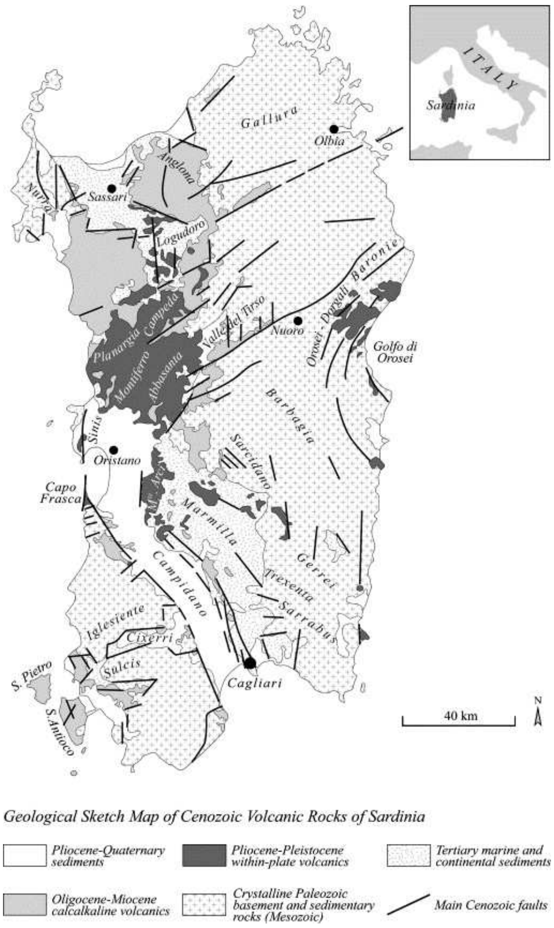


Figure 33 Sardinian lithological complexes

The knowledge of mineralogical components and rocks structure is elemental in order to understand their behavior when used in civil engineering field. (Bini & Scesi, 1983).

They are macroscopically divided according to:

- Identification of the components (minerals, fossils, various included)
- Dimensions and spatial arrangement of the components
- Identification of physical and chemical characters (hardness, fissile, acid reaction, etc.)

The main purpose of this chapter is to present a general knowledge of the mining activity in Sardinia, showing which are the mines still open, detecting the extracted materials and understanding the criteria used for the selection of these materials that are object of study. Among the materials explored, the Granite has not been taken into consideration on purpose even though this is the most popular material extracted in Sardinia and used all over the world. Sardinia has more than ten different types of granite and they are mainly divided by color weaving and composition. That is why granite should be object of a deep study to determine its applicability in road construction.

7.2. The mining history of Sardinia

The exploitation of mineral resources in Sardinia and their processing, have their roots since ancient times: the origin is traced to the Neolithic period with the obsidian extraction. The mining activity of Sardinia began to grow, however, at the time of Roman domination in 238 B.C. The fall of the Western Roman Empire and the subsequent domination of the Vandals and the Byzantines, and the frequent incursions of the Arabs, led to a period of stagnation of mining activity. Only after the 1000 it restarted the interest in the Sardinian mines. The most significant period in the history of mining in Sardinia begins with the birth of the Kingdom of Sardinia in 1720 and during the rule of Savoy. Along with the economic growth in the European continent with its constant demand for raw materials, the Island attracted high foreign and Italian capital. This led to many mining companies, most with non-Sardinian capital. Until the First World War production continued to grow, afterwards the impossibility of exporting minerals to Germany, France, England and Belgium brought to a crisis of the Sardinian mines, to which followed the 1929 crisis with the collapse of the U.S. stock market as well. The Sardinian mines developed once again in the thirties with the autarchic regime of the fascist period, which together with the increasing need for metal weapons, led to the resumption of mining activity, even of those uneconomic abandoned mines. The same post-war reconstruction could not disregard the mining industry.

In the '50s, thanks to the importance of the extractive industry, Sardinia showed a high level of industrialization, however, in contrast to its image of an agro-pastoral region. Until the mid-60s, despite of ups and downs, the mines were a leading business of the Sardinian economy. Since the '60s, the Sardinian mining industry has therefore undergone deep changes, moving from the traditional production of metal ores (lead, zinc) to the current situation in which prevail industrial minerals and ornamental rocks.

Nowadays, mining activities in Sardinia occupy an important role at regional and national levels, as in the past. Comparing to other regions, Sardinia has a fruitful specialization in the extractive sector, both in terms of activities of the territory and employees.

Sardinian mining companies active in 2005 and registered in the Commercial Register of the Chamber of Commerce were 241, 0.2% of the total regional businesses. The majority of them (99.2%) is in the extraction of non-energy minerals field and the 95.9% represents the non-metallic mineral percentage which also includes building stones and ornamental ones.

The category of building stones has the first place for the number of companies registered in the Companies Register in 2005 with 74.7% of the regional mining company (Persico, 2007).

7.3. Material And Selection Procedure

To know what type of materials are extracted in Sardinia and their location it was established the "Regional Cadastre of mine deposits and Public Registry of mining licenses" currently available.

The " Regional Cadastre of mine deposits and Public Registry of mining licenses" contains:

- Summary of active and decommissioned mines in Sardinia; recap of recovered, retrained and renaturalised ex mines and demographic balance of mining activity from 1989 to 2006;
- List of active mines according to different sort order;
- List of inactive mines according to different sort order;
- Recap of regional mining grants by administrative status and activity cultivation status;
- List of current mining grants according to different sort order;
- List of closing and filed mining grants according to different sort order .

Lists of active mines have following information for each mine: Label (ID of the mines in the cartography), name, Municipality where the mine is mainly located, administrative status, year the mining activity started, date the mining authorization was released, expiring authorization date, intended use of the material, main commercial product, lithological material, consistency of stocks, surface of the authorization to the activity license (Ha), space occupied by the mining activity, holder of the mining and possible operator. The first selection parameter chosen is to use the "stock of material in years". Only active mines have been considered to guarantee the production of the material for at least 10 years.

The Table 19 show the list of extracted materials by the selected mines:

Table 19 List of Extracted Material in Sardinia

Province of Cagliari		Province of Sassari	
Material	**Commercial Product**	**Material**	**Commercial Product**
Scisto	Inerti per ril_riemp_str	Calcare	Inerti per conglomerati
Metarenaria	Inerti per ril_riemp_str	Trachite	Trachite di Banari
Metacalcari	Inerti per ril_riemp_str	Basalto	Inerti per conglomerati
Metargilliti	Inerti per ril_riemp_str	Andesite	Inerti per ril_riemp_str
Scisto sericitico	Inerti per ril_riemp_str	Pomice	Materiali per isolanti
Depositi alluvionali	Inerti per conglomerati	Calcare-Marna	Granulati per leganti
Argillite	Granulati per leganti	Scisto Filladico	Granulati per leganti
Calcare	Inerti per ril_riemp_str	Ignimbrite	Inerti per ril_riemp_str
Arenaria	Inerti per conglomerati		
Calcare marnoso	Granulati per leganti		
Andesite	Inerti per conglomerati		
Basalto	Inerti per conglomerati		
Quarzite	Inerti per conglomerati		
Province of Olbia-Tempio		**Province of Carbonia-Iglesias**	
Material	**Commercial Product**	**Material**	**Commercial Product**
Porfido	Inerti per ril_riemp_str	Calcare	Inerti per conglomerati
		Trachite	Trachite di Perdaxius
Province of Nuoro		**Province of Oristano**	
Material	**Commercial Product**	**Material**	**Commercial Product**
Gneiss	Lastrato di Siniscola	Basalto	Inerti per conglomerati
Micascisto	Lastrato di Lula	Ignimbrite	Trachite di Fordongianus
Basalto	Inerti per conglomerati	Trachite	Trachite di Fordongianus
Calcare	Marmo di Orosei	Depositi alluvionali	Inerti per conglomerati
Dolomia	Inerti per conglomerati	Perlite	Materiali per isolanti
Province of Medio Campidano		**Province of Ogliastra**	
Material	**Commercial Product**	**Material**	**Commercial Product**
Depositi Alluvionali	Inerti per conglomerati	Depositi Alluvionali	Inerti per ril_riemp_str
Calcare	Inerti per conglomerati	Calcare	Inerti per ril_riemp_str

In South Africa the natural aggregates that are used for inverted pavement are Dolerite or Tillite. In Sardinia these types of rocks are not available due to the geology of the region, but as shown in the table above, other hard rocks are widely present.

Considering South Africa classification of rock types for road construction purpose, the chemical and physical features of the dolerite, the selected materials available in the region taken into consideration from the table above are Andesite, Quarzite Basalt and Trachite of Serrenti.

The Andesite studied derives from the site of Sarroch and is the one on the south of Sardinia: it is an extrusive igneous rock of intermediate chemistry from afanitica to porphyritic texture. The minerals in the rock are mainly plagioclase, pyroxene and hornblende. Andesite is considered the corresponding outpouring of diorite. Andesite is tipycal of subduction zones such as the western coast of South America. Andesites are erupted at temperatures between 900 and 1100 ° C.

Quartzite is a type of metamorphic rock composed almost exclusively of quartz, and derives from the site of Sinnai. Arising from the dismantling and subsequent passage of metamorphic quarzoareniti. The varieties most colorless, transparent and glassy-looking are composed almost exclusively of quartz. Quartz can be associated with other minerals such as mica, feldspar, plagioclase, carbonates. Given the stability of quartz (mineral not easily alterable), information on the metamorphic grade are given precisely by accessory minerals which may be contained in the quartzite. Texture is mainly massive, but the presence of mica can turn it into shale.

Basalt is the most common rock on the earth surface; it belongs to the large group of extrusive igneous rocks and it is the product of the consolidation of a magma that grows in the depths of the earth, and comes to the surface through ducts and crevices of the volcanic structure. Wide areas of Sardinia are covered by volcanic formations, especially in the northern and west-central sides.

Basalt is an effusive rock of volcanic origin, black or dark-colored with a relatively low silica content. Basalt is mainly composed by calcic plagioclase and pyroxene, and some basalts can also be rich in olivine. The basalt may present with appearance ranging from porphyritic to microcrystalline to glassy.

Regarding the "Trachite", there are many materials that fall under this name, it is actually the term with which it can be classified a volcanic rock; the best term for the trachite Sardinia is Pyroclastics. The trachite of Serrenti, that we have analyzed, is the result of highly explosive volcanic eruptions; this comes from volcanic structures in which heterogeneous mixtures of magma, rock fragments and gas give rise to burning clouds that can either rise in the atmosphere or get down along the flanks of the volcano.

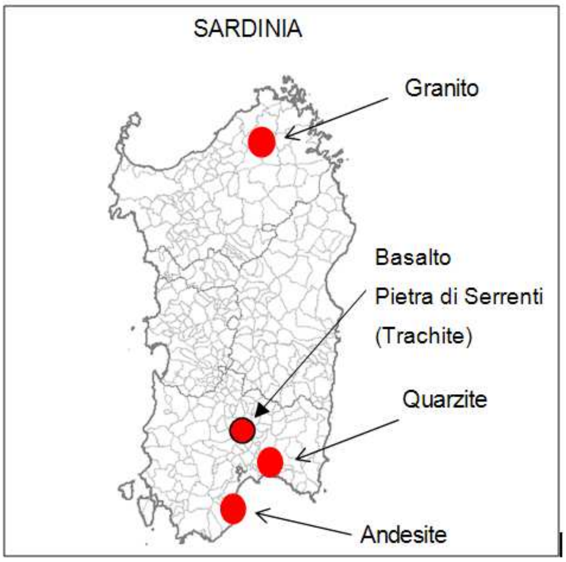


Figure 34 Localization of materials.

8. CHEMICAL - PETROGRAPHYCAL CHARACTERIZATION

8.1. Introduction

This thesis aims to value the possibility of using the available materials to build the infrastructure with the technique of Inverted Pavement. From the chemical-physical point of view, European and South African tests are carried following the same techniques and results are easy to compare. In this chapter chemical and physical properties of the selected materials were analyzed and the results were compared to those of the South African rocks in order to evaluate if these materials had similar physical and chemical characteristics, such as to allow their use in the construction of road pavements using the technique of Inverted Pavement.

8.2. Chemical composition

Tests were performed at the laboratories of Science and Chemical Department of University of Cagliari. The Inductively Coupled Plasma with Optical Emission Spectrometry (ICP-OES) technique was used in order to determine detailed information about the chemical composition. In the Table 20 it is possible to evaluate the chemical composition with the percentages for each component this results are compared with those of dolerite.

Table 20 Chemical Composition of selected materials

ELEMENTS	BASALT (%)	ANDESITE (%)	STONE OF SERRENTI (%)	QUARZITE (%)	DOLERITE (%)
SiO_2	52,30	57,32	61,27	96	49,95
Al_2O_3	12,41	15,0	13,97	1,1	12,78
TiO_2	2,27	0,64	0,84	---	3,51
MgO	3,32	2,0	0,37	0,4	6,59
MnO	0,10	----	0,06	---	0,20
CaO	7,54	9,0	4,35	0,4	9,70
Na_2O	7,09	1,8	4,98	0,8	2,52
K_2O	4,07	1,2	7,54	---	0,91
P_2O_5	0,57	0,4	0,22	---	0,29
SO_3	0,05	2,9	0,05	---	----
Fe_2O_3	10,27	6,0	5,65	1,2	13,12
LOI	----	3,3	0,48	0,4	----
TOT.	99,99	99,56	99,78	100	99,57

It is possible to observe that the basalt is the material that is closest in terms of chemical composition to dolerite: six of the eleven components show a pattern compatible with the percentages of the minerals present in the dolerite. The components of the basalt closest to the values of dolerite are the SiO_2 with a value of 52.30% versus the 49.95% of the Dolerite, l'Al_2O_3 has 0.4% less than the dolerite, the TiO_2 with 2.27% versus 3.51% of the dolerite. A greater difference is found in the value of MgO in which basalt is presented with a smaller percentage of 3.32% versus 6.59% of dolerite; MnO presents the values almost equal; still 3% differences is found in Fe_2O_3, the basalt is 10,27% vs. 13,12% of the Dolerite.

The Andesite has some compatibility only in the case of sodium oxide, with a percentage of 1.8% compared to 2.52% of the dolerite, the potassium oxide 1.2% compared to 0.9% of the Dolerite and of calcium oxide present with 9% in the basalt and with 9.7% of the dolerite. The Trachite of Serrenti show a lower chemical compatibility as you can see only the phosphorus pentoxide has a value of 0.22% in Trachite of Serrenti against the 0.29% of the dolerite. Quarzite is the one that has the lowest chemical compatibility. The chemical composition in terms of percentage of components are shown in the histogram in the Figure 35.

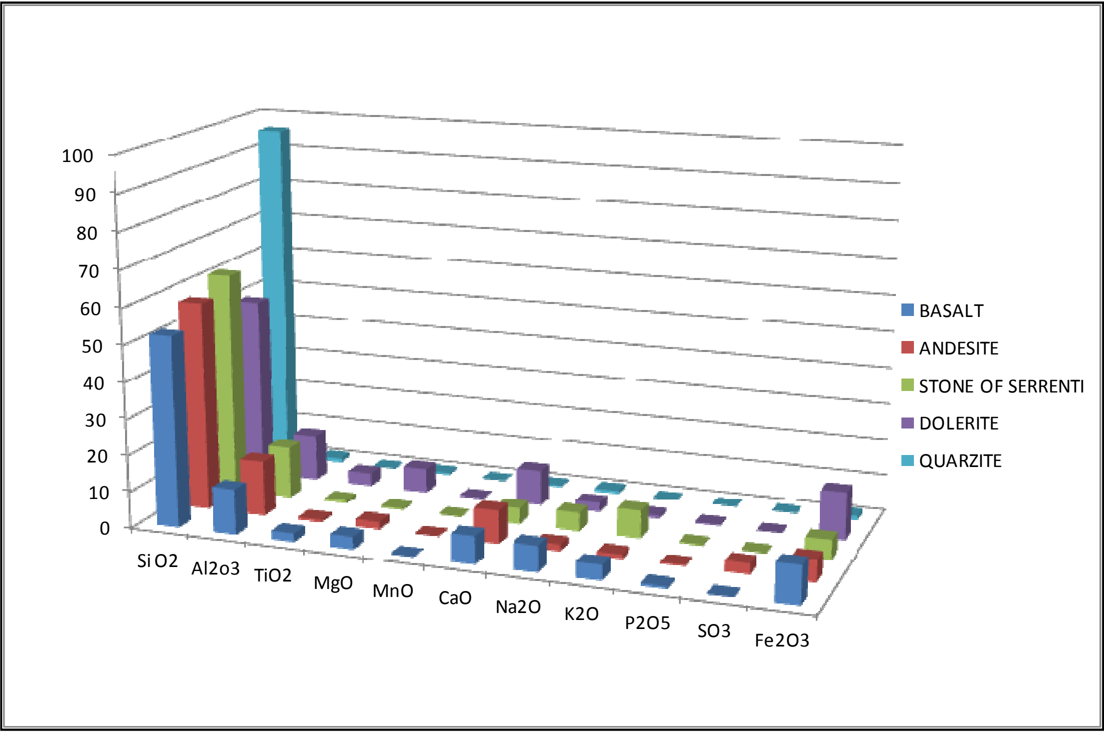


Figure 35 Parallel on the chemical analysis of basalt, andesite, trachite of Serrenti, quarzite and Dolerite.

The calculation of Correlation Index shows better the compatibility of each selected material with Dolerite considered as a reference.

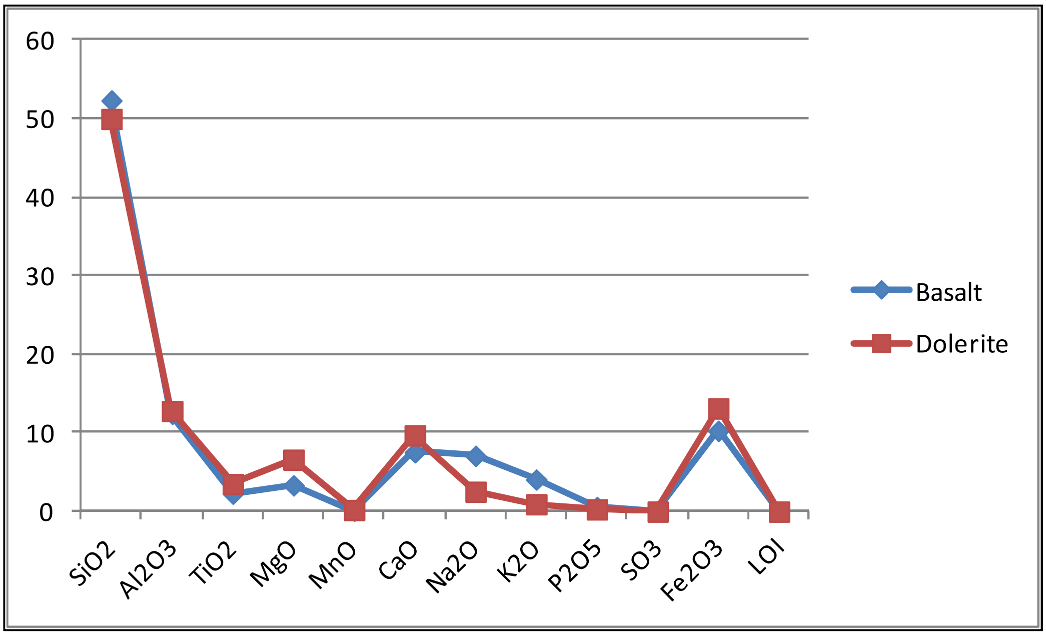


Figure 36 Correlation Index Basalt/Dolerite = 0,986

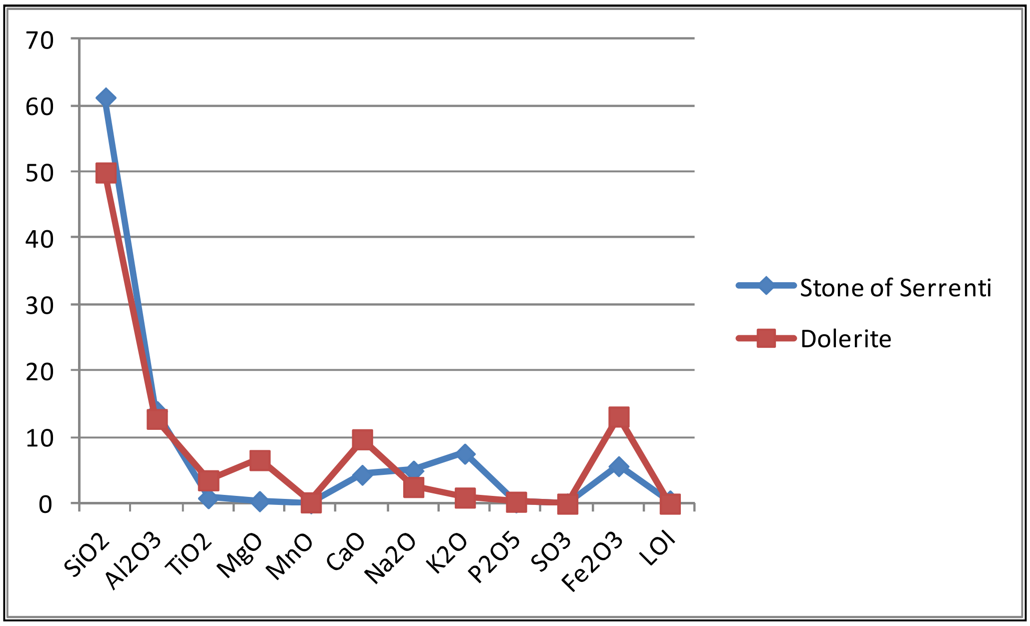


Figure 37 Correlation Index Stone of Serrenti/ Dolerite = 0,962

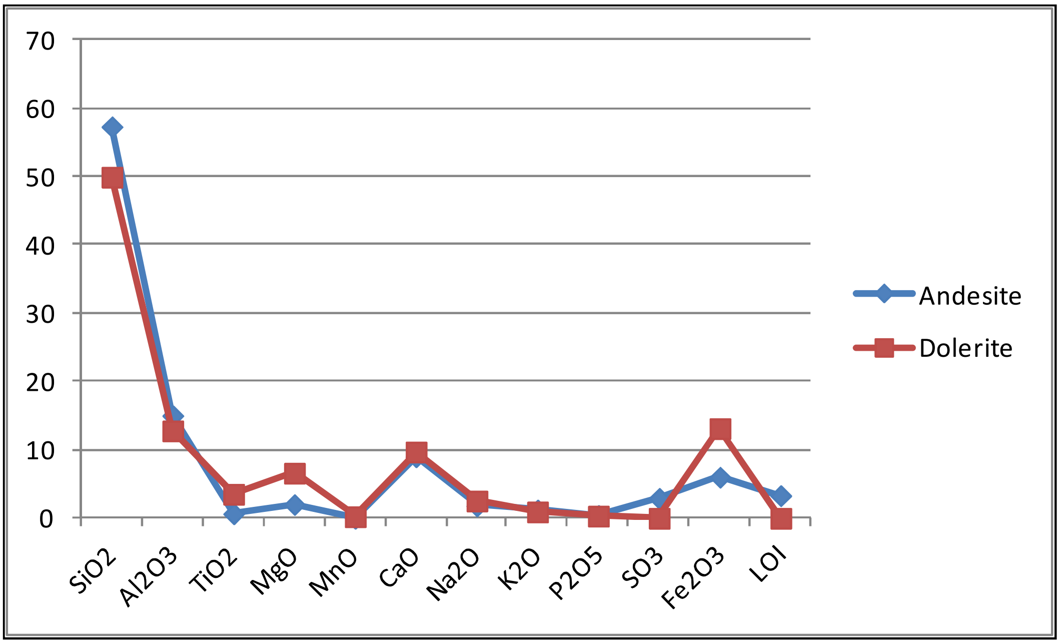


Figure 38 Correlation Index Andesite/Dolerite = 0,976

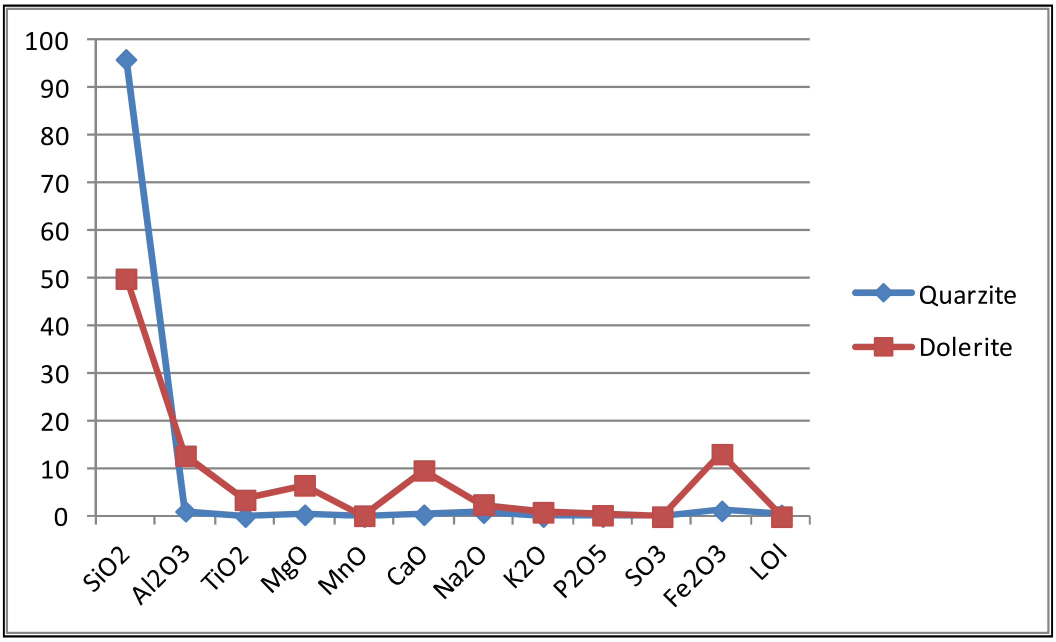


Figure 39 Correlation Index Quarzite/Dolerite = 0,94

At the conclusion of the petrographic chemical-physical surveys, were analyzed the thin sections of the materials that were selected.

8.3. Petrographycal Carachterization

8.3.1. Andesite Thin sections

The thin section is made of volcanic rock characterized by a clearly porphyritic structure with zoned plagioclase phenocrysts (Pl), sometimes of big size (more than 2-3 mm), subordinated anphiboles (Orneblende: Orn) and clinopyroxene (Cpx) of smaller size, absorbed in a ipoialina paste (partially vitreous), dominated by plagioclase phenocrysts. Fitting mineral are made by plenty of a micro phenocrysts of matt oxids (Fe Ox), not only widespread as primary minerals in rocks, but also present in obvious reaction borders all around the orneblende. Mainly the rock clearly presents secondary alteration phenomena in the primary stage; main visible phenomena are the alteration of the essential minerals (Calcic plagioclase, Anphibole, Pyroxene) in Chlorite, sericiti and clay. These phenomenologies are amenable to a hydrothermal alteration of propilitic type. Finally the textural/structural characteristics of the rock let to classify it as Andesite.

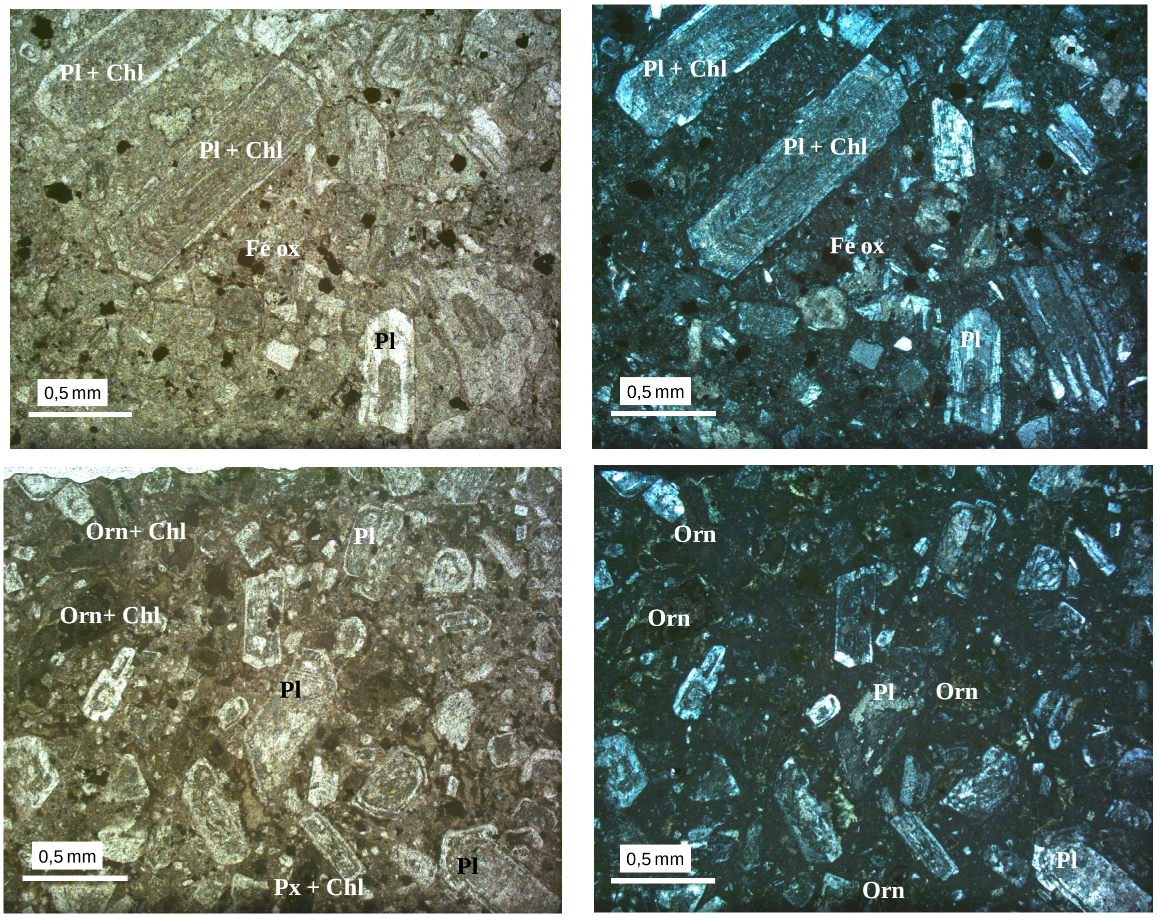


Figure 40 Andesite Thin Sections

From these thin section it is possible to say that this Andesite is very much chloritized, this is why it has cementing power found in other test.

Chloritization is a hydrothermal process of ferromagnesi minerals such as Biotite, Amphibole, Phyroxene with chlorite formations. Green color is given by chlorite. Andesite is a volcanic rock, the consolidation of magma in:

- Phenocrysts: (big minerals 0,2) are Plagioclase
- Bottom Paste: very fine minerals in which Phenocrysts are absorbed.

Bottom Paste suffered a propilitic alteration and it is now of a chloritical base. In Figure 40 are clearly seen Plagioclase (big minerals) absorbed in the bottom paste. Black spots are Iron Oxides.

In the picture of Andesite clinophyroxene are surrounded by a black border of Iron Oxide. The alteration process brings phyroxene which is ferrous to become Iron Oxide. From the study of Thin Sections of the Andesite, the rock has obvious phenomena of secondary alteration of the primary stages; the main phenomena are observable alteration of key minerals (calcic plagioclase, amphiboles, pyroxene) in Chlorites, Sericiti and Clays.

For these reasons, emerged a tendency to assume a plastic behavior on Andesite were not executed the remaining tests as prescribed from South Africa specification.

8.3.2. Basalt Thin Section

The petrography analysis shows a magmatic litotype of grey color, homogeneous and compact, fine-grained, with emphatic porosity all over the surface and locally confused. The main rock constituents are: feldspars (45%) characterized by dimensions which are usually under 0,6 mm and they constitute the bottom mass of the material. Locally are present phenocrysts of 1,6 mm with wretch aspect, superficially altered, zoned and twins. Olivine (20%) in catwalks with a maximum size of 1 mm. Individuals of bigger size suffered a total or partial alteration process. Abundant Iddingsite (5%) of a dark red color, as result of olivine alteration in coronitical structures or in full replacement of them.

Matt minerals are like crystal individuals of Magnetite with maximum size of 0,3 mm or like thin catwalks that roil the various constituents. Among the minor constituents, probable Inosilicates (phyroxene) and Calcite correlated to the alteration process.

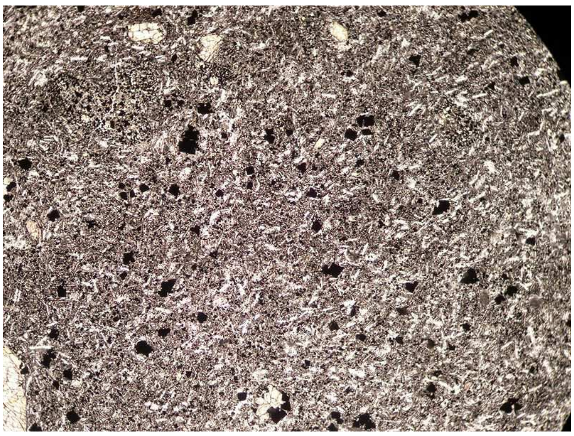


Figure 41 Basalt Thin Sections

8.3.3. Trachite (Stone of Serrenti) Thin Sections

The petrography analysis shows a magmatic litotype of grey color with shades conducing to bordeaux and with grey-beige plagues. It has dark-bordeaux and grey lithoclasts of centimeter and millimeter size and millimeter crystal individuals of black and white colors of compact material characterized by the presence of millimetrical pores.

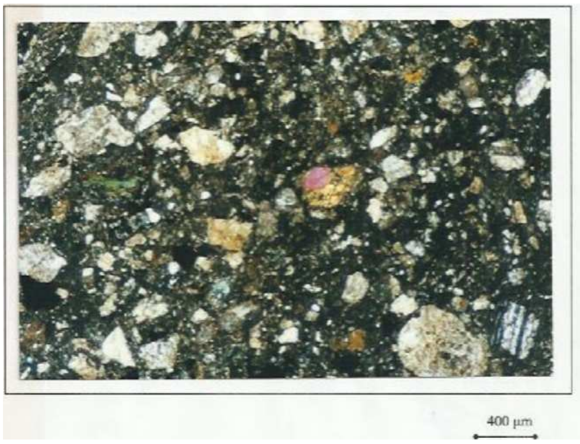


Figure 42 Stone of Serrenti Thin Section

In the Figure 42 we can notice the porphyritic structure of the material. Are visible Feldspati with a whitish color and lithoclasts immersed in a mass with a bottom difficult to identify. The crystal individuals with a yellow, green and light blue color are inosilicate.

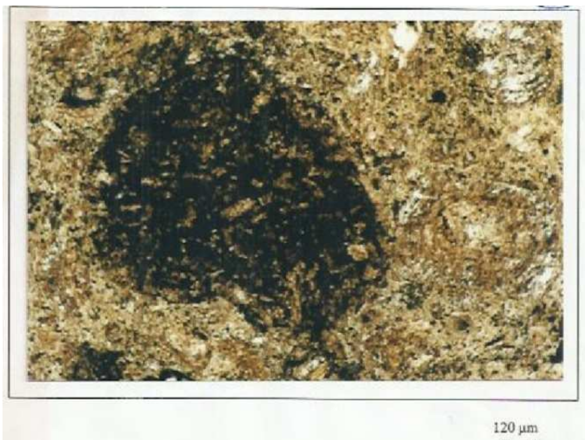


Figure 43 Stone of Serrenti Thin Section

In the Figure 43 we can notice a magma lithoclast composed of crystal individuals weakened by plentiful small masses with a hematite origin. On the right side of the lithoclast, it is possible to glimpse a plagioclase probably divided into zones and difficult to identify given that weakened by the effects of alteration spread on the whole surface.

8.3.4. Quarzite Thin Sections

Quartzite is a rock composed of firmly cemented quartz grains. It results from recrystallization of pure quartz sandstone by metamorphosis (so called metaquartzites). Often quartzites are white or light coloured but they can also be coloured more intensively.

Most quartzites are metamorphic, but exists sedimentary quartzite too, resulting from cementation of quartz sand by silica-rich groundwater solutions (so called orthoquartzites). Mostly light coloured: white, sometimes light gray, yellowish, light brown. Exist also coloured varieties by included minerals: blue, green, purple, or black. Minerals are quartz, small amounts of iron oxides, rutile, mica, carbonates, chlorite, tourmaline needles.

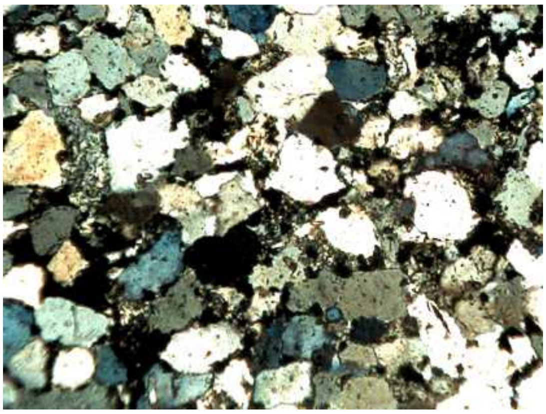


Figure 44 Quarzite Thin Section

9. TRIAXIAL TEST

9.1. Test Procedure

Consolidated drained triaxial tests were performed on Andesite and Quarzite to measure e-modulus and compare it with the values achieved with aggregates used for G1.

The tests were performed with hydrostatic pressure of the cell ($\sigma c= \sigma 3$) equal to, 250 kPa and 100 kPa. These value was chosen based on the calculations performed with the Boussinesq theory, according to which, considering the various types of load provided in the Italian rules and applying such loads statically, the value of the average horizontal pressure on the surface of a base layer of 150 mm varies da σh = 127 kPa, in the case of axle 10 kN (light truck), to σh = 1400 kPa, in the case of axle 110 kN (heavy trucks).

Furthermore, according to Yoder, la $\sigma_h = \sigma_3$ in the sub grade varies between 0 and 150 kPa, so, it is obvious to consider that the solicitations themselves are higher in the base layer.

The cell hydrostatic pressure, shown in the Figure 45, has been applied for 15 minutes during the consolidation phase, before proceeding with the application of the vertical load. The vertical load has been applied through the piston with a speed of 0,66 mm/min.

Figure 45 Triaxial test chamber

The schematization of drainage conditions for a granular soil, because of the insufficient influence of time factor, is not necessary, save in the case, as explained previously, of very rapid solicitations, through fractions of second, where the liquid phase interacts intensely with the solid phase ($u_e > 0$), until causing a downfall (liquefaction).

The size of the specimen varies according to the grading of the material we need to test, although it is always required that the relation between height and diameter of the cylinder is H/D≥2, in order to reduce the effect of the dead zones De Saint Venant including the circling effect of the load plates.

The experimentation has been performed of specimens with diameter $D = 70\ mm$ and height of $H = 150\ mm$.

The obtained data during the tests allowed to elaborate the curves stress-deformations.

The samples were packaged as required by AASHTO Mod. specification, reconstructing the grading curve with different percentages sieve to obtain a sample G1 and considering only the passing at 20mm sieve as required for keeping the right connection between maximum size of the aggregate and the sizes of the triaxial cell. ($d_{max} \leq D/10$). What must be kept in mind is that the elastic modulus obtained is that of a compacted granular mixture through ASSTHO modified, grading according to the curve G1, but it is not an elastic modulus of one G1 layer, because the samples in the laboratory not have followed the complex process of compaction that the material has in the construction site (Slushing).

The tests conducted on Andesite not have allowed to determine the elastic modulus of the sample, or rather the elastic modulus obtained has no meaning in this study, but further test are needed to better understand this behavior.

The standard Elastic-modulus value used for G1, using Dolerite, is normally of 300 MPa when is laid over a granular layer; higher values are achieved when laid over a cemented natural gravel layer named in South Africa C3.

The plastic behavior of andesite evident in the thin section was confirmed also by the triaxial test, where it was not possible to find the breaking point and, consequently, not even the Elastic Modulus.

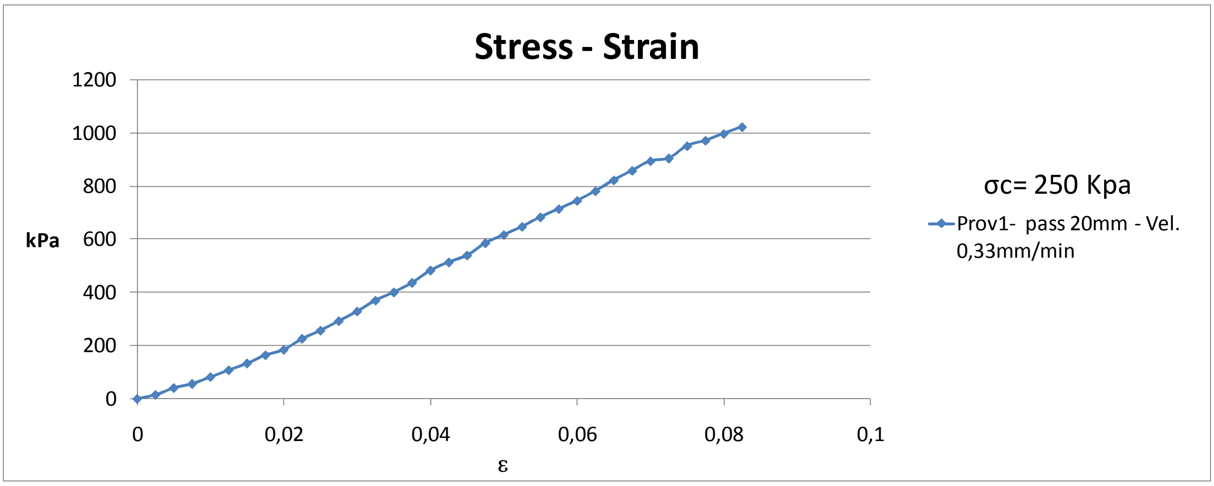


Figure 46 Triaxial test Diagram of Andesite

About Quartzite, the chartes reported in the Figure 47 and in the Figure 48, concerning two different values of the confinement pressure, show the classic elastic-plastic trend.

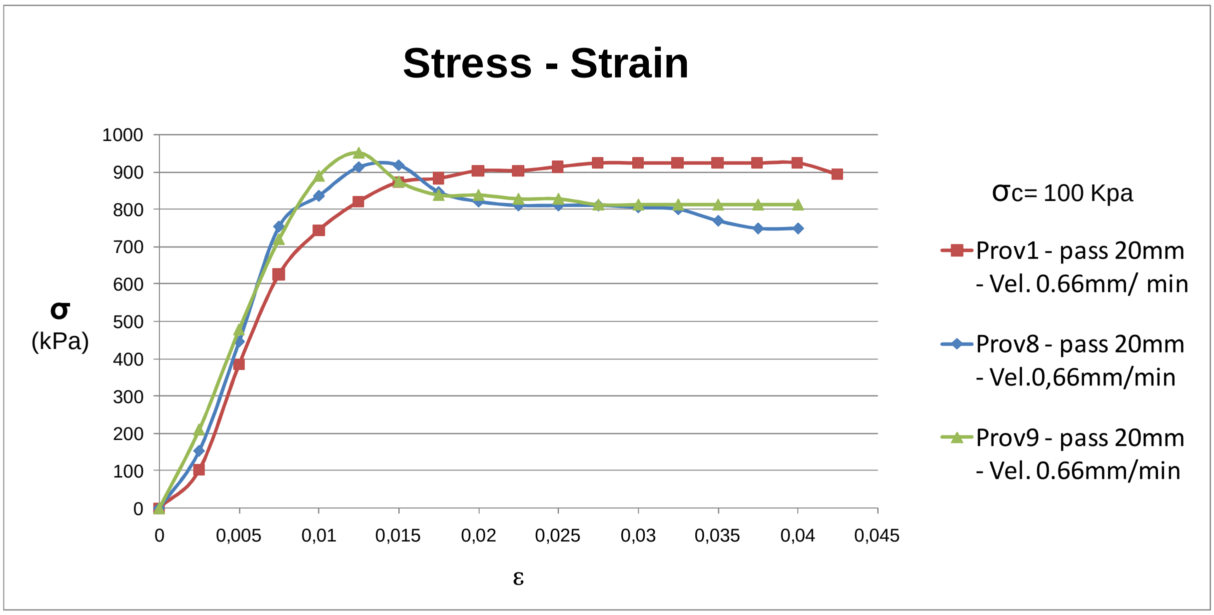


Figure 47 Triaxial test consolidated drained with cell pressure 100 kPa.

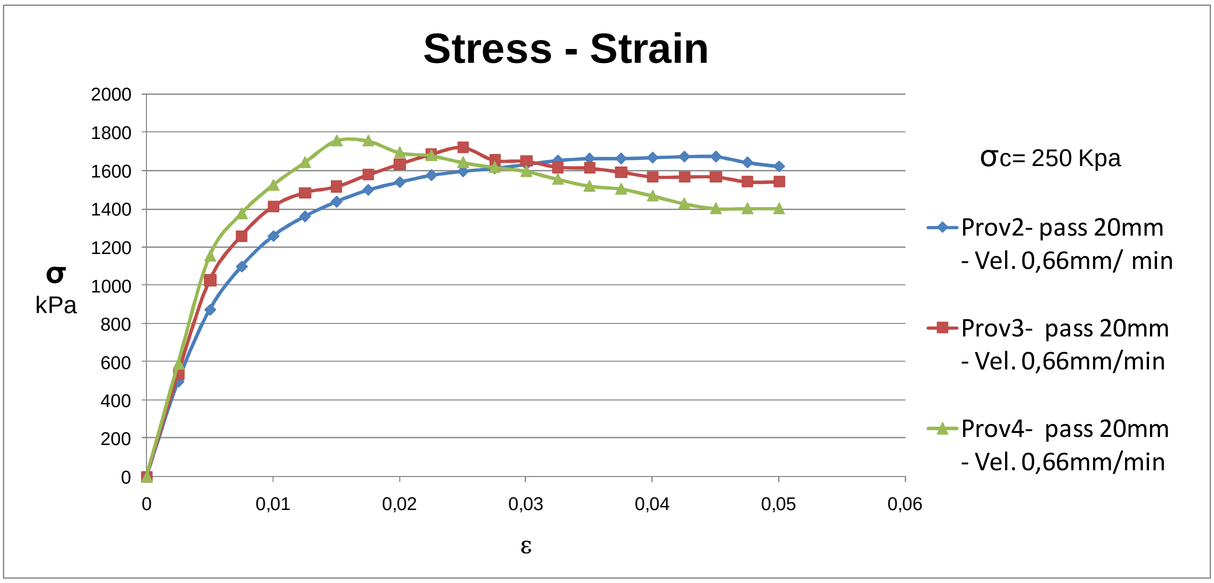


Figure 48 Triaxial test consolidated drained with cell pressure 250 kPa.

After having done the tests, through an analysis of the results, was calculated the Elastic Modulus of each specimen. The values that were obtained are reported in the Table 21.

Table 21 Triaxial Test Results of Quarzite

Sample n°	Confinement Pressure kPa	Density g/cm³	Young's Modulus MPa
1	100	2,26	104,74
2	100	2,25	120,15
3	100	2,25	101,66
4	250	2,34	174,57
5	250	2,33	205,4
6	250	2,32	231,0

10. SOUTH AFRICAN PHYSICAL- MECHANICAL LABORATORY TEST

Anyway it is not enough to comparison chemistry and petrography to establish that a material is not suitable, since other characteristics of a physical mechanical might ensure a good yield in the realization of the base layer super compacted. For this purpose, were analyzed physical parameters such as density, compressive strength, porosity, absorption coefficient and mechanical strength as the first parameter of the coefficient obtained from the test Los Angeles. The material that we want to use must actually have good characteristics of compression strength and abrasion resistance related to the strong energy compaction imprinted by compacting machines; it must also have reduced absorption coefficients and porosity, as during the phases of paving layer base are used large amount of water (Slushing process). The results of the tests are presented in the Table 22.

Table 22 Results of physical-mechanical properties.

Tests	Basalt	Andesite	Stone of Serrenti	Quarzite	Dolerite
Density: [kg/m³]	2478-2624	2490–2662	2204-2688	2400-2660	2800-3100
Uniaxial compressive Strength [MPa]	133	91	93	122	40
Porosity: [%]	3,9	5.56	17	5,5	1
Water Absorption: [%]	1.3	1.236	6.3	0,04	0.28-0.9

Los Angeles Test Values: [%]	14	24.86	18	28	14.5-15.5

Looking at the Table 22 it shows once again the compatibility of basalt in terms of density with an overlap of valuesaround 2800 Kg / m³, porosity index of 4,9% compared to 1% of the Dolerite, and coefficient Los Angeles coincidental. The Quarzite shows a good compatibility in terms of Density, a greater resistance by compression, but an insufficient value of Los Angeles test. The Andesite occurs, as a result of the physical-mechanical test, a valid candidate even if the values obtained from the Los Angeles test of the order of 24,8% are higher than those of Dolerite, while the value of water absorption presents values of 1,2% andesite against an average value of 0,6% of the dolerite. While again the Trachite of Serrenti and that with physical-mechanical characteristics which differ from those we are looking for, despite of a value of Los Angeles of 18% close to 15% of the average dolerite. As it is easy to observe basalt, quartzite, andesite and stone of Serrenti have very high values of uniaxial compressive strength of about 186-91-93 MPa vs 40 MPa of the dolerite.

South Africa unbound granular layers are named by the South African Technical Recommendations for Highways (TRH) (Authorities, 1986) with the acronyms of G1-G10, from high to poor mechanical performances. Dolerite and Tillite rocks are used in South Africa to construct G1 layers. The TRH specifications are written for the practising engineer and describe current, recommended practice in selected aspects of highway engineering. G1 is the best quality material used for specific base course layers and subjected to very strict construction processes with the aim of achieving values of density material from 86% to 88% of apparent relative density, that is about 106%-108% modified Proctor density (Araya A.A., 2011), forming an ultra-compacted layer.

Based on South African regulation for G1 layer this chapter describes laboratory tests to verify the possibility of replacing the base with available material.

Table 23 Crushed Stone Base material Requirements

MATERIAL CHARACTERISTIC		TYPE OF MATERIAL G1	G2	G3	
PARENT MATERIAL		Sound rock from an approved quarry, or clean, sound mine rock from mine dumps, or clean sound boulders	Sound rock, boulders or coarse gravel	Sound rock, boulders or coarse gravel	
ADDITIONAL FINES		Only fines crushed from the same sound parent rock may be added for grading correction provided that added fines shall have a LL not exceeding 25 and PI not exceeding 4.	May contain up to 10% by mass of approved natural fines not necessarily obtained from parent rock. Added fines shall have a LL not exceeding 25 and PI not exceeding 6.	May contain up to 15% by mass of approved natural fines not obtained from parent rock. Added fines shall have a LL not exceeding 25 and PI not exceeding 6.	
STRENGTH		10% Fines Aggregate Crushing Value (10% FACT), determined in accordance with TMH1 method B2, shall be not less than the appropriate value in table 3602/2, column 3. The Aggregate Crushed value (ACV), determined in accordance with TMH1 method B1, shall not exceed the appropriate value in table 3602/3.			
DURABILITY		The material shall comply with the requirements in columns 3, 4 and 5 of table 3602/2.			
FLAKINESS INDEX		Flakiness Index, determined in accordance with TMH1 method B3, shall not exceed 35 on each of the -26,5 + 19 mm fraction and the -19 +13,2 mm fraction.			
FRACTURED FACES		All faces shall be fractured faces.	For crushed materials at least 50% by mass of the fractions retained on each standard sieve 4,75 mm and larger shall have at least one fractured face.		
ATTERBERG LIMITS	FRACTION (mm) -0,425	LL shall not exceed 25. PI shall not exceed 5. LS shall not exceed 2%. In addition the arithmetic mean of the PI's for a lot (min 6 tests) shall not exceed 4.	LL shall not exceed 25. PI shall not exceed 6. In addition the arithmetic mean of the PI's for a lot (min 6 tests) shall not exceed 4,5. LS shall not exceed 3%.	LL shall not exceed 25. PI shall not exceed 6. LS shall not exceed 3%. In the case of calcrete the PI shall not exceed 8. (% passing 0,425 mm sieve) LS ≤ 170	
	-0,075	The PI shall not exceed 12. If the PI exceeds 12 the material shall be chemically modified. After chemical modification the PI of the minus 0,075 mm fraction shall not exceed 6.		If chemical modification is required, the PI of the -0,075 mm fraction after modification shall not exceed 10.	
SOLUBLE SALTS		See additional requirements.			
NOMINAL MAXIMUM SIZE		37,5 mm	37,5 mm	37,5 mm	26,5 mm
GRADING	Nominal aperture size of sieve (mm)	Percentage passing sieve, by mass	Percentage passing sieve, by mass	Percentage passing sieve, by mass	
	37,5	100	100	100	
	26,5	84 - 94	84 - 94	84 - 94	100
	19,0	71 - 84	71 - 84	71 - 84	85 - 95
	13,2	59 - 75	59 - 75	59 - 75	71 - 84
	4,75	36 - 53	36 - 53	36 - 53	42 - 60
	2,00	23 - 40	23 - 40	23 - 40	27 - 45
	0,425	11 - 24	11 - 24	11 - 24	13 - 27
	0,075	4 - 12	4 - 12	4 - 12	5 - 12
COARSE SAND RATIO (SEE DEFINITION IN SUBSUBCLAUSE 3602(c)(i)(5))		Shall not be less than 35% and shall not exceed 50% in respect of the target grading.	Shall not be less than 35% and shall not exceed 50% in respect of the target grading.	Shall not be less than 35% and shall not exceed 50% in respect of the target grading.	
COMPACTION REQUIREMENTS		Minimum of 88% of apparent relative density.	Minimum of 85% of bulk relative density.	98% or 100% of modified AASHTO density (as specified).	

Table 24 G1 Specification according to TRH14

Tests	Specifications	Unit of Measure	Limits
Liquid Limit	TMH1 method A2	%	Max 25
Plasticity Index	TMH1 method A3	%	Max 4
Linear Shrinkage	TMH1 method A4	%	Max 2
10% F.A.C.T. (dry)	TMH1 method B2	kN	Min 110

10% F.A.C.T. (wet)	TMH1 method B2	% of dry	Min 75
ACV	TMH1 method B1	%	Max 29
Flakiness Index (-26.5,+19.0 & 19.0, +13.2	TMH1 method B3T	%	Max 35
Apparent Density	TMH1 method B14/15	kg/m^3	(88%) 2496

10.1. Grading

Unbound granular base derives its high stability from particle interlock and inter-particle friction. Gradation has a profound effect on material performance.

It might be reasonable to believe that the best gradation is one that produces the maximum density. This would involve a particles arrangement where smaller particles are packed between the larger particles, which reduce the void space between particles. This creates more particle-to-particle contact. A widely used equation to describe a maximum density gradation was developed by Fuller and Thompson in 1907. Their basic equations:

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

Where:

- P = % finer than the sieve
- d = aggregate size being considered
- D = maximum aggregate size to be used
- N = parameter which adjust curve for fineness or coarseness (for maximum particle density n~ 0,5 according to Fuller and Thompson)

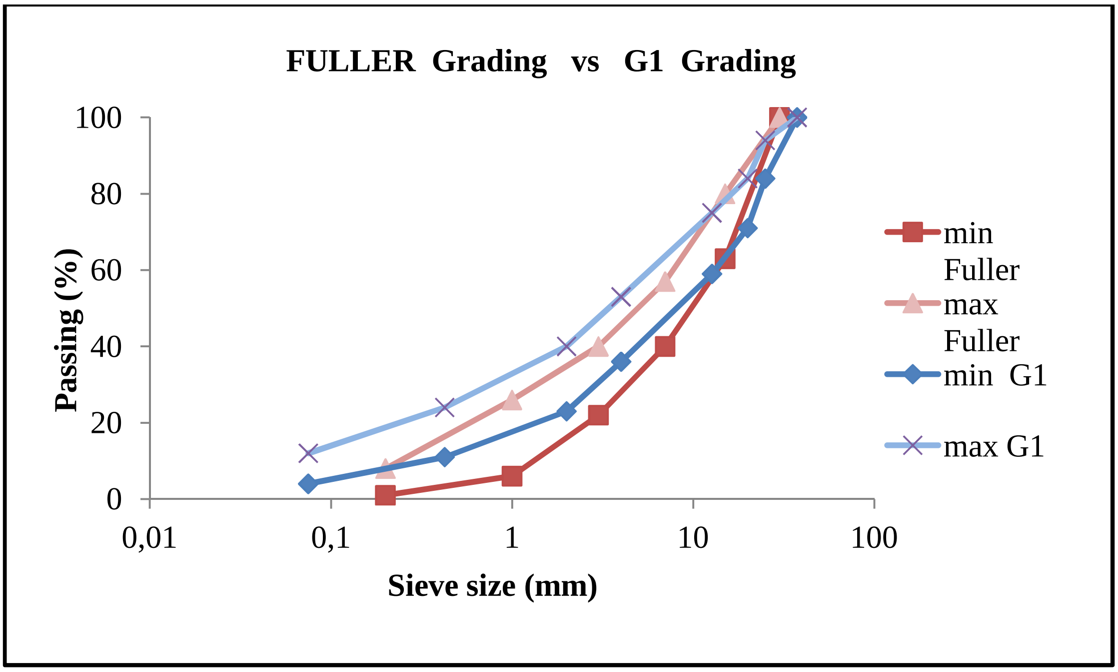


Figure 49 Comparison between Fuller grading curves and G1 base layer grading curves

In the Figure 49 is shown the parallelism between the Fuller curves and the G1 grading curves for Inverted Pavement base layer.

Gradation is very important to geotechnical engineering, it is an indicator of engineering properties such as compressibility, shear strength, and hydraulic conductivity. To obtain a good matrix a clear amount of fines is needed, the knowledge of the amount of the fines percentage and the gradation of the coarse particles is useful to make a choice of material for base courses of highways.

Base layers constructed with crushed stone must possess high resistance to deformation in order to withstand the high pressure imposed upon them. The functions of a base layer are prevention of pumping, drainage, prevention of volume change of sub-grade, increased structural capacity and expedition of construction. To accomplish these functions high density and stability are required.

Grading with little or no fines content (Figure 50a), like course graded, grains stability derive from grain-to-grain contact. Grading that contains no fines usually has a relatively low density but it is pervious and not frost susceptible. However, this material is difficult to handle during construction because of its non-cohesive nature. Grading that contains sufficient fines to fill all voids between the aggregate grains, like medium coarse, will still gain its strength from grain-to-grain contact but has increased shear resistance (Figure 50 b). Its density is high and its permeability is low.

This material is moderately difficult to compact, but it is ideal from the standpoint of stability. As shown in the Figure 50 c, the material that contains a great amount of fines has no grain-to-grain contact and the aggregate merely‘float’ in the soil. Its density is low; it is practically impervious and it is frost susceptible. In addition, the stability of this type of material is greatly affected by adverse water conditions. (Yoder & Witczack, 1975)**.**

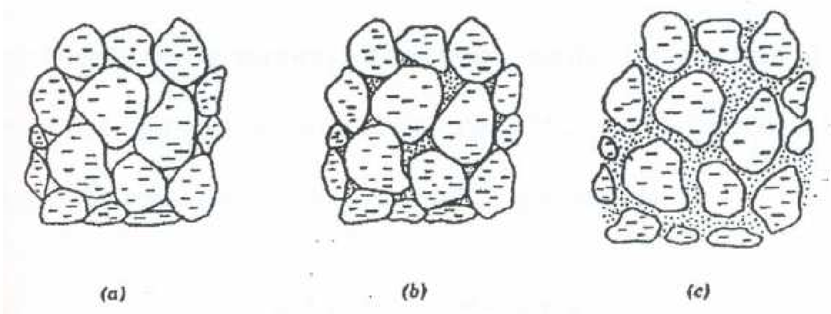


Figure 50 Coarse and fines particles distribution

The Figure 50a indicates the packing state resulting in the largest G/S (grain/Sand) ratio as almost no sand grains to occupy a portion of the voids between the coarse aggregate particles. Mixtures at this state develop shear or permanent deformation resistance primarily by friction resistance between gravel size particles and it may not be very stable.

Grain/Sand ratio decreases when more sand fractions exist until an optimal packing configuration that is reached at the ideal state shown in the Figure 50b. This ideal state means the voids between the gravel size particles are completely occupied by the bulk volume of the sand grains, developing the condition of minimum porosity. The minimum porosity of the mixture can be theoretically interpreted as the boundary between a gravel-controlled and a sand-controlled mixture.

The Figure 50b can also derive that the minimum porosity of the mixture is the product of the porosity of each individual fraction with the same specific gravity assumed for all fractions. After that, if sand fractions keep increasing (or G/S ratio decreases), then packing conditions will dictate gravel (or coarse) particles to “float” in the sand-fine matrix and have trivial control over shear strength behaviour of the mixture (Figure 50c) (Xiao Y. , Tutumluer, Qian, & Siekmeie, 2012).

The increase in permanent deformation with an increase in saturation can be explained by the lubrication of the aggregates, and the development of pore water pressure within the materials that can then result in a reduction in effective stress. The lubrication of the aggregates helps the grains to rotate and slide against each other that then cause further compaction.

10.2. Grading for G1Base Layer

According to TRH 14 G1 must have a specific grading depending on the nominal maximum size of passing the 37.5 mm sieve or 26.5 mm one.

Table 25 Grading of Graded Crushed Stone G1

Sieve size (mm)	**Percentage passing by mass**	
	G1 Nominal maximum size of Aggregate (mm)	
	37,5	**26,5**
53,0	100	100
37,5	100	100
26,5	84 - 94	100
19	71 - 84	85 - 95
13,2	59 - 75	71 - 84
4,75	36 - 53	42 - 60
2,00	23 - 40	27 - 45
0,425	11 - 24	13 - 27
0,075	4 - 12	5 - 12

In the picture there are two grading, 26,5 and 37,5 used for creating the G1 base layer and the grading curve of the sample test which has been used to perform the mechanical tests of this research. As you can see during this work we used grading 26,5.

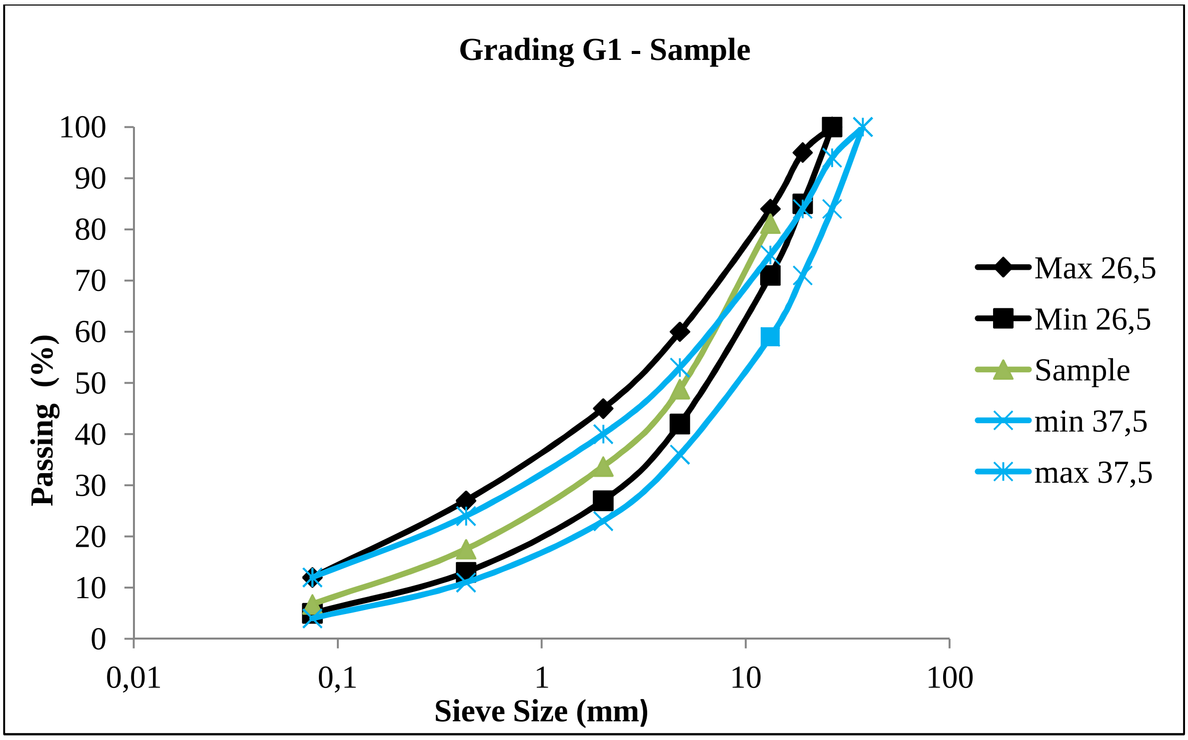


Figure 51 G1 Grading according to TRH14

To understand how far the grading used for the G1 base layer is from the one used for Italian base layers, in the following figure there are both grading and the sample test.

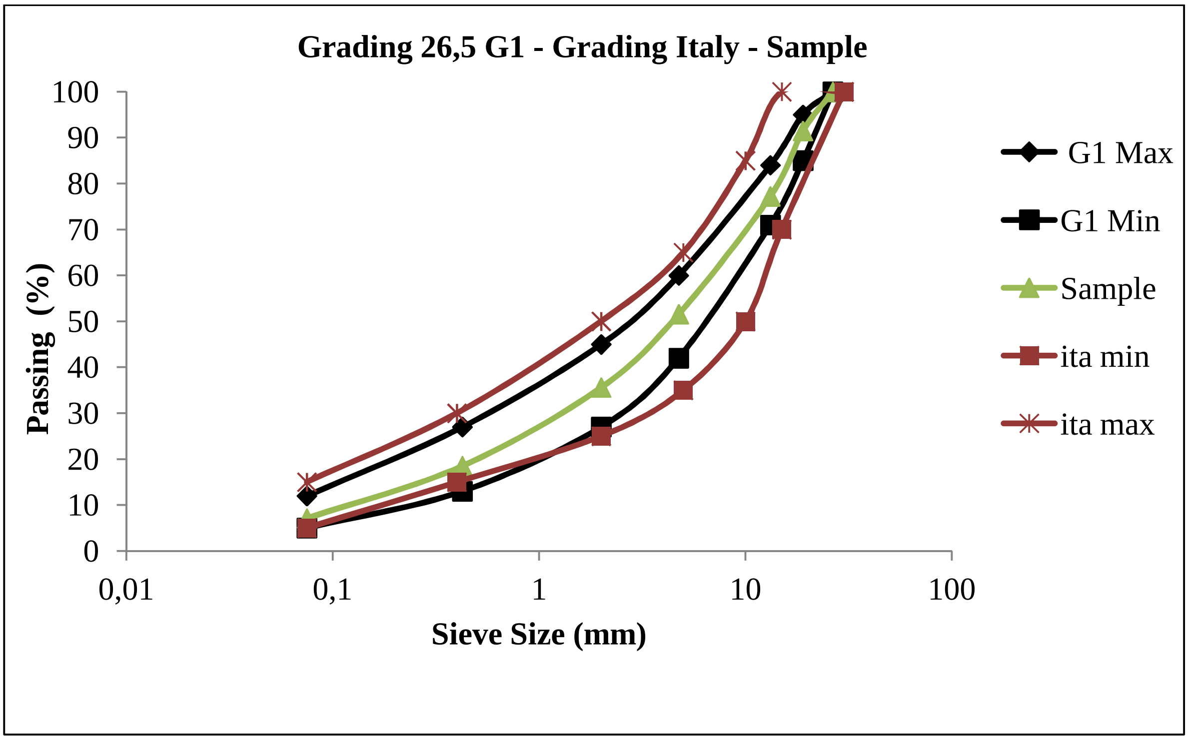


Figure 52 LaGrange Grading

10.3. Maximum Dry Density and Optimum Moisture Content

The maximum dry density and optimum moisture content, as defined below, is determined by establishing the moisture-density relationship of the material when prepared and compacted at the Modified AASHTO compaction effort at different moisture contents.

Maximum density: The maximum density of a material for a specific compactive effort is the highest density obtainable when the compaction is carried out on the material at varied moisture contents. Optimum moisture content: The optimum moisture content for a specific compactive effort is the moisture content at which the maximum density is obtained.

10.3.1. Andesite

In the table below are show the Proctor test results for Andesite. The test was performed following the TMH1 method A7:

Table 26 Proctor Test

Sample	Weight Wet material	Weight Dry material	Water	Mould Volume	Density
	(g)	(g)	(%)	(cm^3)	(g/cm^3)
A	63	59,9	5,17	2472,75	2,0
B	83	76,8	8,07	3179,25	2,2
D	85	77,5	9,67	3179,25	2,4
E	175,5	160	9,68	3179,25	2,3

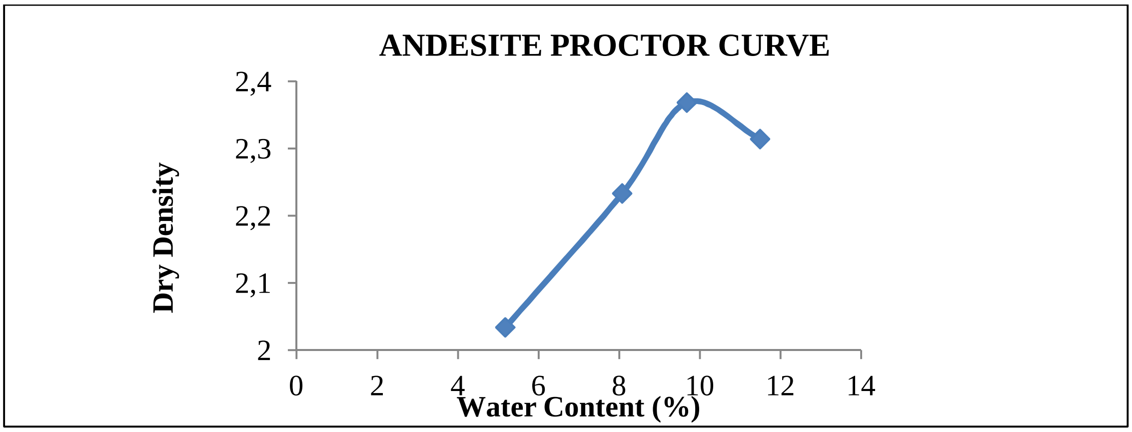


Figure 53 Determination of Optimum Water Content for Andesite material

Maximum Dry Density: 2,4 g/cm^3

Optimum Moisture Content: 9,8 %

10.3.2. Quarzite

In the Table 27 are show the Proctor test results for Quarzite. The test was performed as reported in TMH1 method A7:

Table 27 Quarzite Proctor Test

Sample	Weight Wet material	Weight Dry material	Water	Mould Volume	Density
N°	(g)	(g)	(%)	(cm^3)	(g/cm^3)
1	2172	2109,2	3	943	2,2
2	1482	1404,9	5,5	943	1,5
3	1303	1201,2	8,5	943	1,3
4	2154	2120,5	1,6	943	2,2
5	1413	1399	1	943	1,5

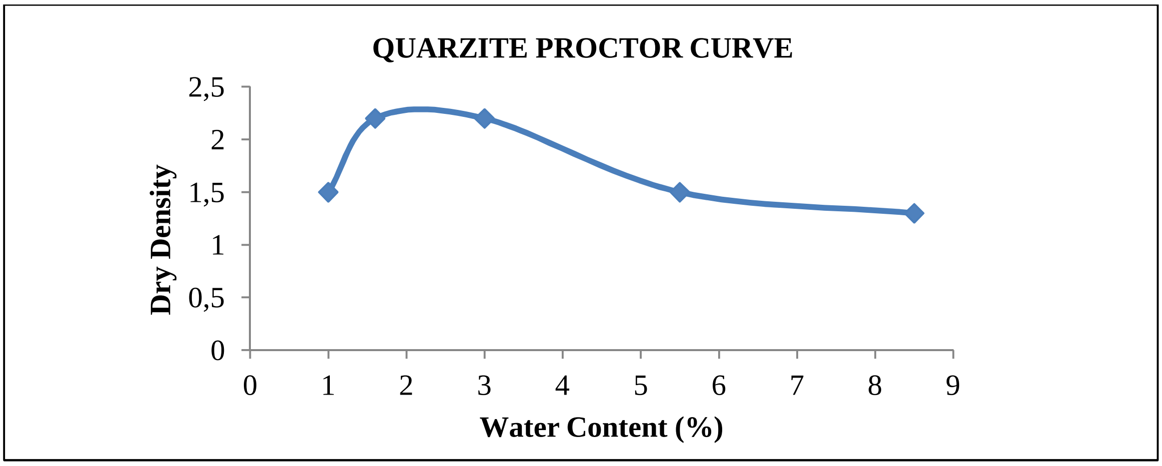


Figure 54 Determination of Optimum Water Content for Quarzite Material

Maximum Dry Density: 2,3 g/cm^3

Optimum Moisture Content: 2,2 %

10.4. Atterberg Limits

Following the method A3 and A2 of TMH1 the determination of plastic limit, plasticity index, and limit liquid were performed.
The determination of the plastic limit is defined by measuring the lowest moisture content at which the soil can be rolled into threads 3 mm in diameter without the threads crumbling, This is expressed as a percentage of the mass of the oven-dried soil, at the boundary between the plastic and semi-solid states.
The liquid limit (LL) is obtained using a Casagrande cup; is conceptually defined as the water content at which the behavior of a clayey soil changes from plastic to liquid.
The plasticity index is the numerical difference between the liquid limit and the plastic limit (PI= LL-PL) of the soil. Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 are non-plastic.

Table 28 Atterberg Limit Results

Tests	Unit	Results ANDESITE	Results TRACHITE	Results BASALT	Results QUARZITE	Limits
Plastic Limit	%	15,52	N.D	N.D	N.D	-
Liquid Limit	%	26,3	N.D	N.D	N.D	Max 25
Plasticity Index	%	10,8	N.D	N.D	N.D	Max 4

The results show that the Andesite one does not meet the limits required by plasticity, and for this reason we will not perform the remaining tests required by the South African regulations.

10.5. Fines Aggregate Crushing Value (10% FACT)

The 10 per cent Fines Aggregate Crushing Value is defined as the force in kN required to crush a sample of -13,2+9,5mm aggregate so that 10 per cent of the total test sample will pass a 2,36 mm sieve.
As described in SABS Method 841, at least three test specimen (each of mass about 3 kg) are required for each test (wet and dry).

Table 29 10% FACT dry test results of Quarzite

Sample	Weight material	Max Load	Passing at 3,35 mm	Note	10%FACT Dry Value	Limits (min)
N°	(g)	(kN)	(g)		(kN)	(kN)
1	2858	314	145	20mm,10 min	654	110
2	2905	607	261	30mm,10 min		
3	2953	2150	865	40mm,10 min		

10% FACT Wet Test: 75% of Dry test =490 kN

Table 30 10% FACT dry test results of Basalt

Sample	Weight material	Max Load	Passing at 3,35 mm	Note	10%FACT Dry Value	Limits (min)
N°	(g)	(kN)	(g)		(kN)	(kN)
1	2990	110	176	15mm,10 min	194	110
2	2890	180	261	20mm,10 min		
3	3076	213	437	30mm,10 min		

10% FACT Wet Test: 75% of Dry test = 145,5 kN

Table 31 10% FACT dry test results of Trachite

Sample	Weight material	Max Load	Passing at 3,35 mm	Note	10%FACT Dry Value	Limits (min)
N°	(g)	(kN)	(g)		(kN)	(kN)
1	2955	153	204	15mm,10 min	152	110
2	2890	174	239	20mm,10 min		
3	2905	225	456	30mm,10 min		

10% FACT Wet Test: 75% of Dry test = 114 kN

As we can see from the tables above, the results fully meet the regulatory limits.

10.6. Aggregate Crushing Value (ACV)

The ACV of an aggregate as defined hereunder is determined by crushing a prepared confined aggregate sample under a specified, gradually applied compressive load and determining the percentage of the material crushed finer than a specified size.

The aggregate crushing value (ACV) of an aggregate is the mass of material, expressed as a percentage of the test sample which is crushed finer than a 2,36 mm sieve when a sample of aggregate passing the 13,2 mm and retained on the 9,50 mm sieve is subjected to crushing under a gradually applied compressive load of 400 kN. The compression test machine capable to applying a load of 400 kN and which can be operated at a uniform rate of loading so that this load is reached in 10 minutes. The ACV test includes a dry test and wet test.

Table 32 ACV dry test results of Quarzite

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits (max)
N°	(g)	(kN)	(g)	(%)	(%)
1	2109	400	233	10,57	29
2	2080	400	259	12,45	29
3	2099	400	247	11,77	29

Table 33 ACV wet test results of Quarzite

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits (max)
N°	(g)	(kN)	(g)	(%)	(%)
1	2048	400	318	15,53	29
2	1990	400	270	13,57	29
3	2120	400	360	16,98	29

Table 34 ACV dry test result of Basalt

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits ACV
N°	(g)	(kN)	(g)	(%)	(%)
1	2240	400	432	19,29	29
2	2134	400	512	23,99	29
3	2263	400	572	25,28	29

Table 35 ACV wet test results of Basalt

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits (max)
N°	(g)	(kN)	(g)	(%)	(%)
1	2180	400	588	26,97	29
2	2097	400	597	28,47	29
3	2210	400	510	23,08	29

Table 36 ACV dry test results of Trachite

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits (max)
N°	(g)	(kN)	(g)	(%)	(%)
1	2110	400	253	11.99	29
2	2234	400	297	13,29	29
3	2099	400	321	15,29	29

Table 37 ACV wet test results of Trachite

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits (max)
N°	(g)	(kN)	(g)	(%)	(%)
1	2050	400	309	15,07	29
2	2118	400	360	16,99	29
3	2106	400	374	17,75	29

As we can see from the tables above, the results fully meet the regulatory limits especially for Quarzite.

11. MY EXPERIENCE AT GEORGIA INSTITUTE OF TECHNOLOGY

The Georgia Institute of Technology of Atlanta, in collaboration with the Georgia Department Of Transportation (GDOT), since ‘90s, studies the Inverted Pavement technique (IP). In the spring of 1999, GDOT engineers became familiar with IP technology while attending an international transportation symposium in South Africa. In November 1999, GDOT met with the Georgia Construction Aggregate Association (GCAA), to discuss rising construction costs and the need to pursue alternative, cost-effective construction materials and methods. The IP system was introduced, and the GCAA expressed much interest.

The meeting was followed by the construction of an IP test section, utilized as an access road at the Lafarge Building Materials quarry near Madison, Georgia (Morgan County).

In 2009, thanks to the experience reached, was realized the second test section in the village of LaGrange. This second test section is part of an industrial parkway intended to serve the growing car manufacturing industry in South -west Georgia.

Test sections have been continuously monitored in terms of traffic entity and in terms of deformation, rutting and cracking. This data is being provided to support IP as a potentially viable alternative pavement choice for roadways in Georgia.

Since March 2015 until August 2015 I spent a period of research at the Georgia Institute of Technology (Georgia Tech), during which I examined in depth the knowledge on the IP technique, and together with my supervisor the Doc Frost and GDOT, I examined the test sections of LaGrange and Morgan County. During the visits to test sections, I collect samples of the material used for realizing test section and, when they were brought at laboratories of the Georgia Institute of Technology, I did the tests considered by the South African Regulations. The purpose of tests done is that of understanding if the rock used has good mechanical features, such that they can be used in the realization of a infrastructure, through the Inverted Pavement technology.

11.1. Morgan County test section

Morgan County test section was realized as an access way towards a mine of granite, therefore:

- traffic is of heavy loads kind
- the material used for infrastructure is Granite.

The experimental test section was structured in this way:

<u>- 1° section</u>: 300 meters of **Conventional Section**

- <u>2° section</u>: 120 meters of **Inverted Pavement South Africa**

- <u>3° section</u>: 120 meters of **Inverted Pavement Georgia**

The entire road required as much as 1.8 m of fill, which consisted of mostly waste aggregate, To ensure that the load carrying capacity of the sub-grade met a minimum California Bearing Ratio (CBR) value of 15, 100% of granitic aggregate was used. An additional 51 mm of graded aggregate base (GAB) was placed so that all three sections would have a consistent sub-base. The first section, beginning at Seven Islands Road, was called the "conventional" section and was completed using 150 mm of compacted surge stone followed by 200 mm of compacted GAB. The remaining 244 m was divided into two, 122 m IP test sections. Both IP sections where completed with a 200 mm cement treated base (CTB) layer and 150 mm GAB layer. The only difference between the two IP sections was in the construction of the GAB: the first section was constructed with the "slushing" method; the second section using conventional construction methods. Both the conventional and IP sections were overlaid with 75 mm of 19 mm Superpave hot mix asphalt (HMA). As you can see in the figure below:

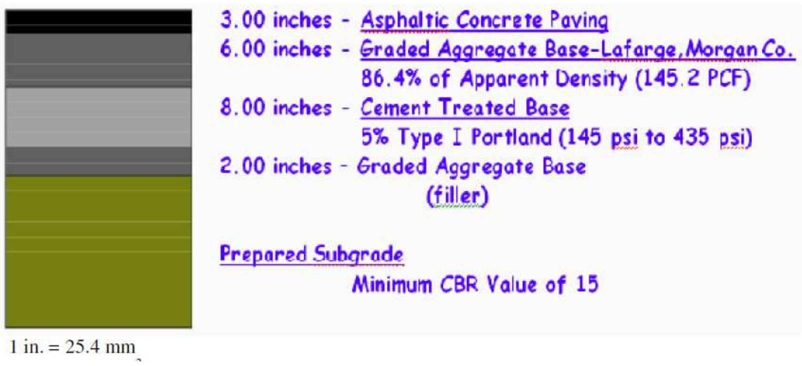


Figure 55 Inverted pavement Cross-section: Morgan County

The results reached during about 12 years, allowed to prove how, through materials different to those South African and through variations of the procedures of laying, their infrastructure reacts in an excellent conditions. The monitoring of all the deforming phenomena was done in December 2003 and then in November 2006, identifying sections that are far 15 metres each other and in 7 various points of pavement, as in the outline reported in the Figure 56 and in the Figure 57:

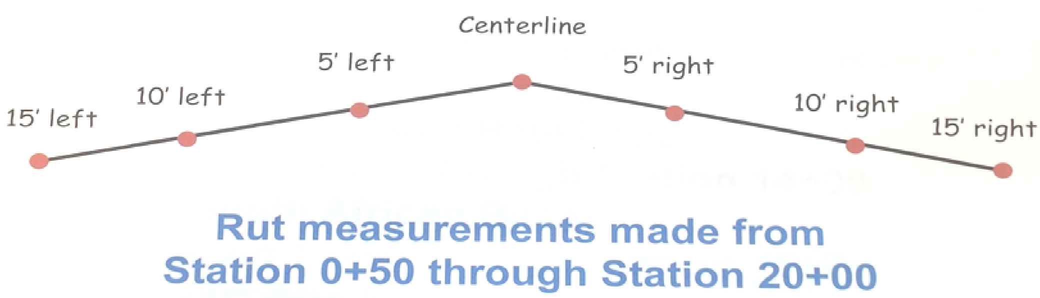


Figure 56 Scheme of monitoring section

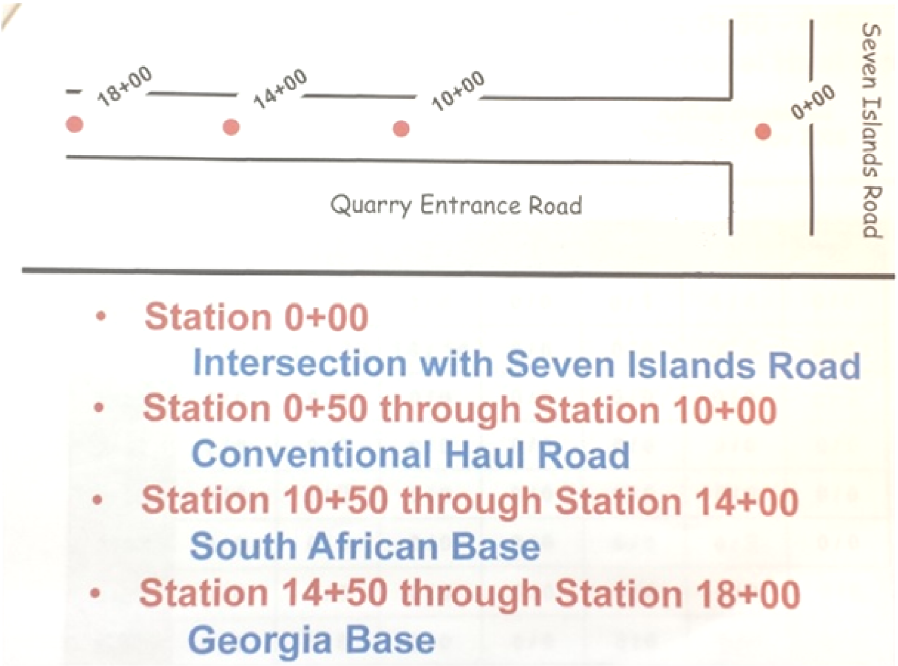


Figure 57 Test section scheme

The left side of pavement is identified as the lane of exit from quarry; instead, the right side is identified as the lane of access to quarry. Measurements report the depth of the cracks present in pavement at the same point, at a distance of 3 years the first measurement compared to the second. The unit of measure is 1,6 mm. It was possible to observe immediately how the lane of exit from quarry, where trucks travel loaded, is more cracked than the lane of access. The results of measurements are illustrated in the Table 38:

Sta. #	15 ft. Left of C/L (2003/2006)	10 ft. Left of C/L (2003/2006)	5 ft. Left of C/L (2003/2006)	Centerline (C/L) (2003/2006)	5 ft. Right of C/L (2003/2006)	10 ft. Right of C/L (2003/2006)	15 ft. Right of C/L (2003/2006)
0+50	0/0[1]	5/5	0 / 8	0/0	0/1	4/4	0/0
1+00	0/16+	16/16	14/14	0/0	0/0	3/3	0/0
1+50	0/0	0/0	0/ 0	0/0	0/0	0/0	0/2
2+00	0/0	0/0	0/ 0	0/0	0/0	0/0	0/0
2+50	0/0	0/2	0/0	0/0	0/0	0/0	0/0
3+00	0/0	0/0	0/0	0/0	0/0	0/2	0/0
3+50	0/0	0/0	0/0	0/0	0/0	0/0	0/0
4+00	0/0	0/0	0/0	0/0	0/0	0/0	0/0
4+50	0/0	0/0	0/0	0/0	0/0	0/0	0/0
5+00	0/0	2/2	0/0	0/0	0/0	0/0	0/0
5+50	0/0	0/4	0/0	0/0	0/0	0/0	0/0
6+00	0/0	0/3	0/0	0/0	0/0	0/0	0/0
6+50	0/2	0/4	0/0	0/0	0/1	0/0	0/0

Table 38 Results of Conventional Section

The same thing is possible to be observed for the test section realized with the South African Inverted Pavement. I highlighted the deterioration that, during three years, lane was subjected to: from 0 cracking to 3 cracking units, etc. as we can see in the Table 39, where emerges immediately that the more deteriorated side is the external side where weighs down the weight of loaded truck, while it exits from mine.

Sta. #	15 ft. Left of C/L (2003/2006)	10 ft. Left of C/L (2003/2006)	5 ft. Left of C/L (2003/2006)	Centerline (C/L) (2003/2006)	5 ft. Right of C/L (2003/2006)	10 ft. Right of C/L (2003/2006)	15 ft. Right of C/L (2003/2006)
10+50	0/3[1]	0/0	0/0	0/0	0/0	0/0	0/0
11+00	0/4	3/6	0/0	0/0	0/0	0/0	0/0
11+50	0/0	1/1	0/0	0/0	0/0	0/0	0/0
12+00	0/1	0/0	0/0	0/0	0/1	0/0	0/0
12+50	0/3	0/1	0/0	0/0	0/0	0/0	0/0
13+00	0/2	0/1	0/0	0/0	0/1	0/0	0/0
13+50	0/0	0/2	0/0	0/0	0/0	0/0	0/0
14+00	0/0	0/0	0/0	0/0	0/0	0/0	0/0

Table 39 Test Results of Inverted Pavement South Africa

Table 40 Test results of Inverted Pavement Georgia

Sta. #	15 ft. Left of C/L (2003/2006)	10 ft. Left of C/L (2003/2006)	5 ft. Left of C/L (2003/2006)	Centerline (C/L) (2003/2006)	5 ft. Right of C/L (2003/2006)	10 ft. Right of C/L (2003/2006)	15 ft. Right of C/L (2003/2006)
14+50	0/0	0/0	0/0	0/0	0/0	0/0	0/0
15+00	0/0	0/0	0/0	0/0	0/0	0/0	0/0
15+50	0/0	0/0	0/0	0/0	0/0	0/0	0/0
16+00	0/0	0/0	0/0	0/0	0/0	0/0	0/0
16+50	0/0	0/0	0/0	0/0	0/0	0/0	0/0
17+00	0/0	0/0	0/0	0/0	0/0	0/0	0/0
17+50	0/0	0/0	0/0	0/0	0/0	0/0	0/0
18+00	0/0	0/0	0/0	0/0	0/0	0/0	0/0

No deterioration was observed in the Inverted Pavement Georgia.

The rutting observed in the two IP sections was insignificant; however, minor and *major* rutting was found within the conventional section, especially in the eastbound lane, in which the haul trucks are loaded. Rutting levels over 1 in. (25 mm) were measured at the quarry gate where trucks stop. No cracking in the asphaltic concrete layer was observed in the IP test sections during observations on 3rd December 2003 and on 13th November 2006. However, Extensive cracking, however, was observed in the conventional haul road section. Most of the extensive cracking was located in the eastbound lane, and advanced deterioration was observed where loaded trucks were stopping at the quarry gate.

11.1.1. Falling Weight Deflectometer Testing

FWD testing was conducted in November, 2007 in the Morgan County. This testing was conducted to examine how the Inverted Pavement (IP) sections compared with the conventional haul road section. At each drop location, two seating drops, at a target load of 7,000 lbf, were used, and twelve (12) recorded drops (three each at target loads of 7000 lbf (3150 kg), 9000 lbf (4050 kg), 11000 lbf (4950 kg) and 16000 lbf (7200 kg)).
Based on the deflection data (Figure 58 and Figure 59) gathered for the loading above, the IP sections were determined to be in an excellent condition. After the analysis of the deflection data collected the Remaining Life (RL) as a percentage of the original pavement life, were calculated and reported in the Table 41. (Lewis, Ledford, Georges, & Jared, 2012)

Table 41 Remaining Life

Section	Remaining Life (%)
Gerogia IP	99,34
Sud Africa IP	94,61
Conventional	67,92

The average RL determined for the Georgia IP was 99.34% of the original design life, 94.61% for the South African IP, and 67.92% for the conventional haul route pavement.

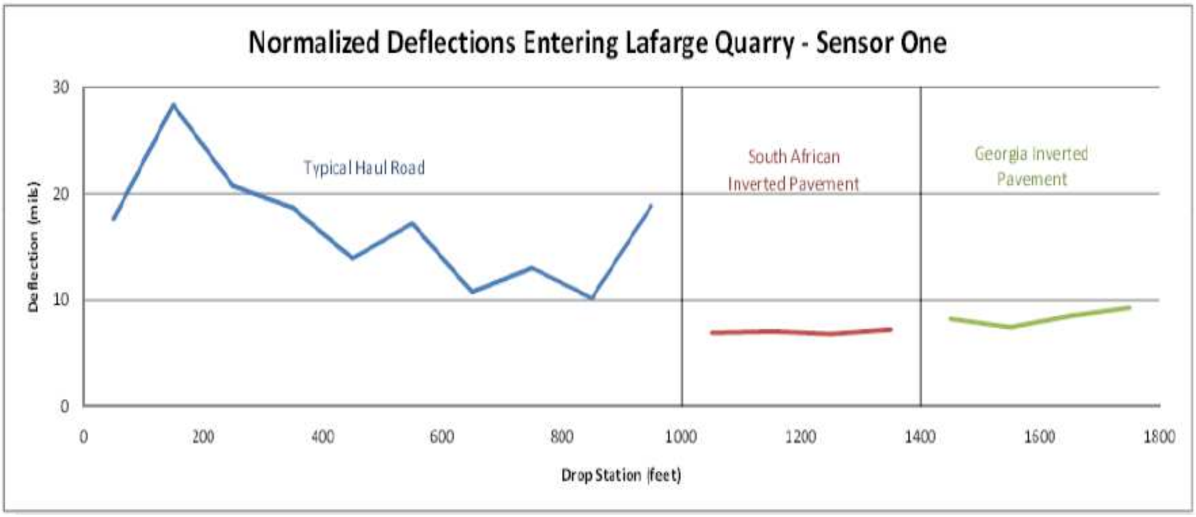


Figure 58 Normalized Deflections Entering Lafarge Quarry

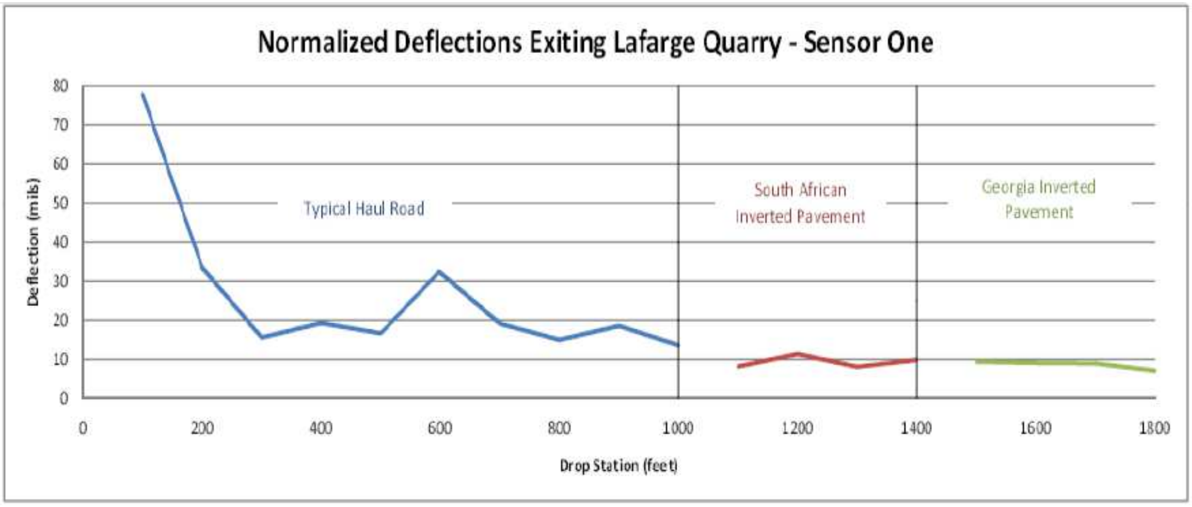


Figure 59 Normalized Deflections Exiting Lafarge Quarry

11.2. LaGrange test section

Based on the success of the Morgan County Project (MCP), and due to the rising cost of asphalt cement, GDOT decided to fund and build its own IP test section on the South LaGrange Loop in Troup County. This project, hereafter referred to as the LaGrange Bypass Project (LBP), was one of several for the preparation of the Kia automotive plant being built in the neighbouring town of West Point, Georgia, and it connects the Kia plant with a newly developed industrial park. The two-lane, 3.24 km roadway was constructed with a conventional, portland cement concrete (PCC) pavement structure, except for the 1047-m IP test section, as you can see in the Figure 60. Construction of the LBP began in January 2008, including the IP test section. (Lewis, Ledford, Georges, & Jared, 2012)

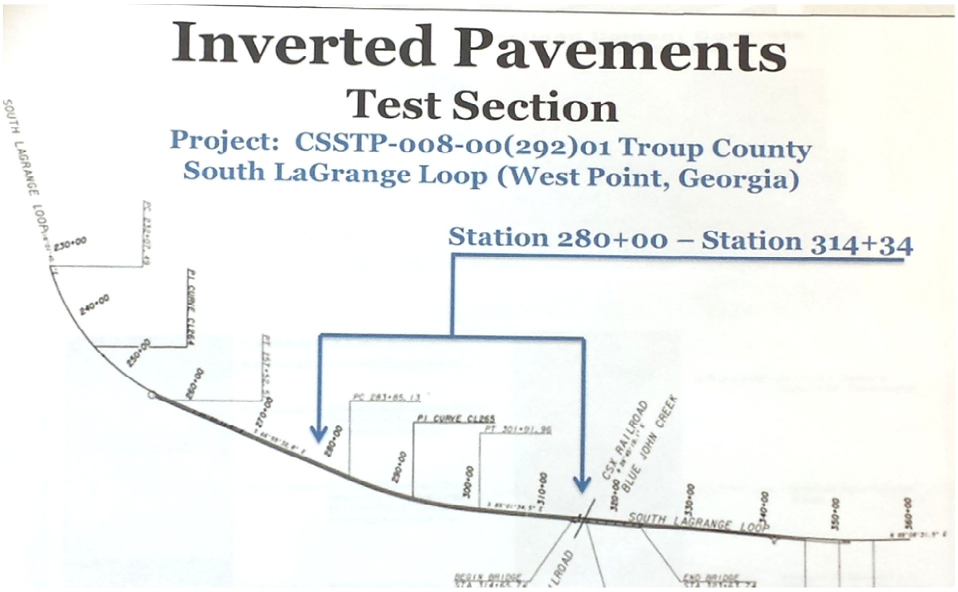


Figure 60 Test Section scheme of LaGrange

Construction of the LBP began in January 2008, including the IP test section, which consisted of (1) stabilized sub-grade; (2) cement-treated base (CTB); (3) graded aggregate base (GAB); and (4) HMA. The sub-grade, CTB, and GAB courses were constructed in accordance with GDOT Specifications Special Provision 320 (Inverted Pavements). The IP and PCC cross sections on the LBP are shown in the Figure 61:

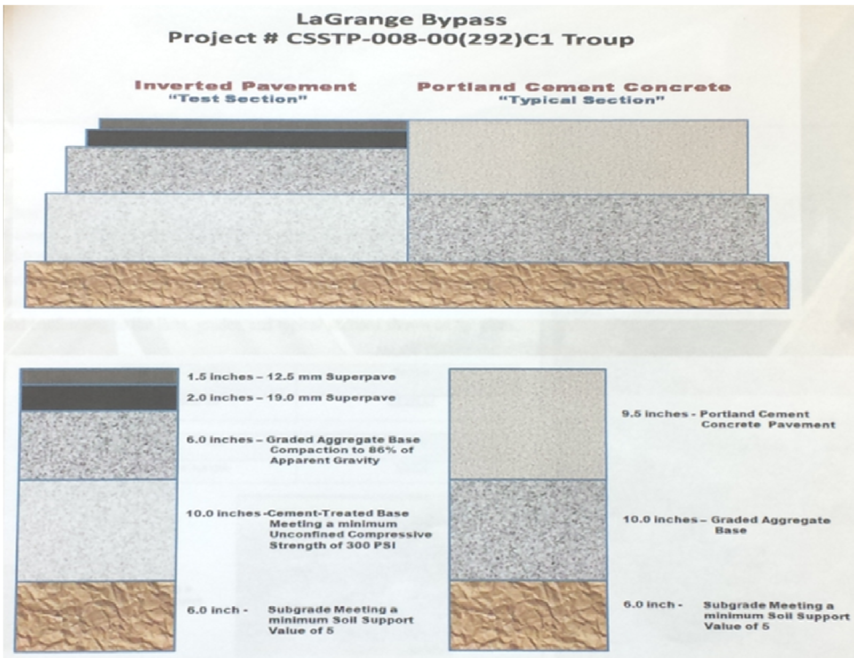


Figure 61 LaGrange scheme Layers of Inverted Pavement and Typical section

Since its completion in 2009, the LBP has been structurally evaluated. Between 20th August 2008, and 14th May 2009, FWD testing was conducted along the length of the newly constructed roadway to assess the strength of the newly placed pavement layers. The new pavement structure evaluated as a whole was determined to be in an excellent condition with an average normalized deflection of 8.54 mils (0,22 mm).

11.3. Characterization Material of Morgan County and LaGrange

During the period of research at the Georgia Institute of Technology were done the main laboratory tests considered by the South African Regulations, for understanding if materials used in the tests sections have the mechanical features required by the South African Regulations to be used in the realization of IP.

The material used for the realization of the LaGrange test section is granite, and it comes from the quarry called "Vulcan". For the Morgan County test section, being the way of access to the quarry of granite, was used the material mined from the quarry itself. The main tests done on samples are the 10% FACT (Fines Aggregate Crushing Value), and the ACV test (Aggregate Crushing Value).

The ACV of an aggregate is determined by crushing a prepared confined aggregate sample under a specified, gradually applied compressive load and by determining the percentage of the material crushed finer than a specified size.

The aggregate crushing value (ACV) of an aggregate is the mass of material, expressed as a percentage of the test sample which is crushed finer than a 2,36 mm sieve when a sample of aggregate passing the 13,2 mm and retained on the 9,50 mm sieve is subjected to crushing under a gradually applied compressive load of 400 kN. The compression test machine is capable to applying a load of 400 kN and which can be operated at a uniform rate of loading so that this load is reached in 10 minutes. The ACV test includes a dry test and a wet test.

The 10 per cent Fines Aggregate Crushing Value is defined as the force in kN required to crush a sample of -13,2+9,5mm aggregate so that 10 per cent of the total test sample will pass a 2,36 mm sieve.

As described in SABS Method 841, at least three test specimen (each of mass about 3 kg) are required for each test.

11.3.1. LaGrange laboratory test

The first test done on the samples come from the quarry "Vulcan", that supplied the material for LaGrange test section was the 10%FACT. In the Table 42 that follows, are showed the results relatives to three samples.

Table 42 LaGrange 10%FACT results

Sample	Weight material	Max Load	Passing at 2,36 mm	Note	10%FACT Dry Value	Limits min
N°	(g)	(kN/pound)	(g)		(kN)	(kN)
1	3003,2	63/14120	100,2	13mm,10 min	126	110
2	3015,5	108/24410	242,5	20mm,10 min		
3	3043,8	175/39400	458,5	25mm,10 min		

10% FACT Wet Test: 75% of Dry test = 94,5 kN

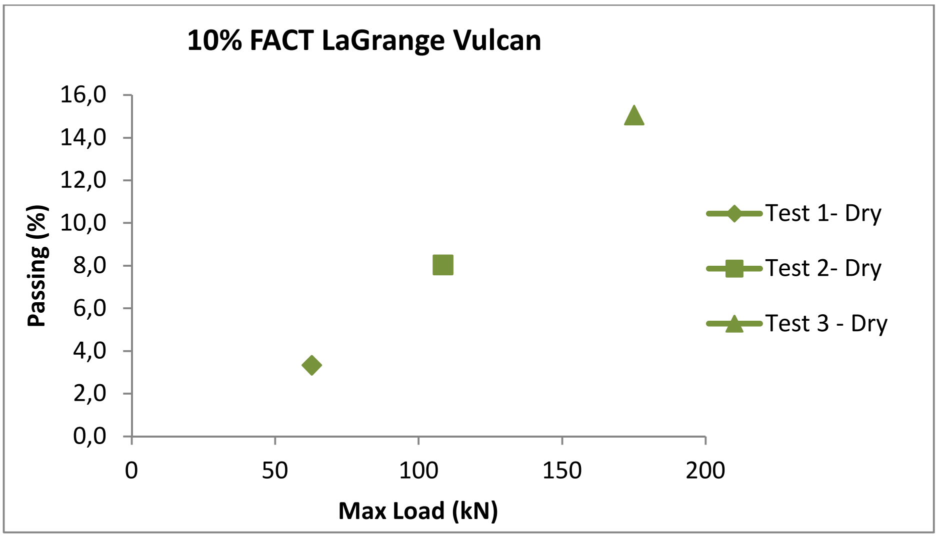


Figure 62 Distribution of 10% FACT results of LaGrange

As shown in table 42 the result of the 10% FACT Wet Test does not meet the minimum value required by the specification.

Then, after having sealed the samples, was done the ACV dry test and then the ACV wet test. the Table 43 and the Table 44 show the results of tests.

Table 43 ACV dry test results of LaGrange

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits (max)
N°	(g)	(kN/pound)	(g)	(%)	(%)
1	1991,6	400/90000	562,1	28,2	29
2	1872,6	400/90000	529,5	28,3	29
3	1902,4	400/90000	541,2	28,4	29

Table 44 ACV wet results of LaGrange

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits (max)
N°	(g)	(kN)	(g)	(%)	(%)
1	1924	400/90000	575,2	29,9	29
2	1985,2	400/90000	585,9	29,5	29
3	1943,6	400/90000	609,9	31,38	29

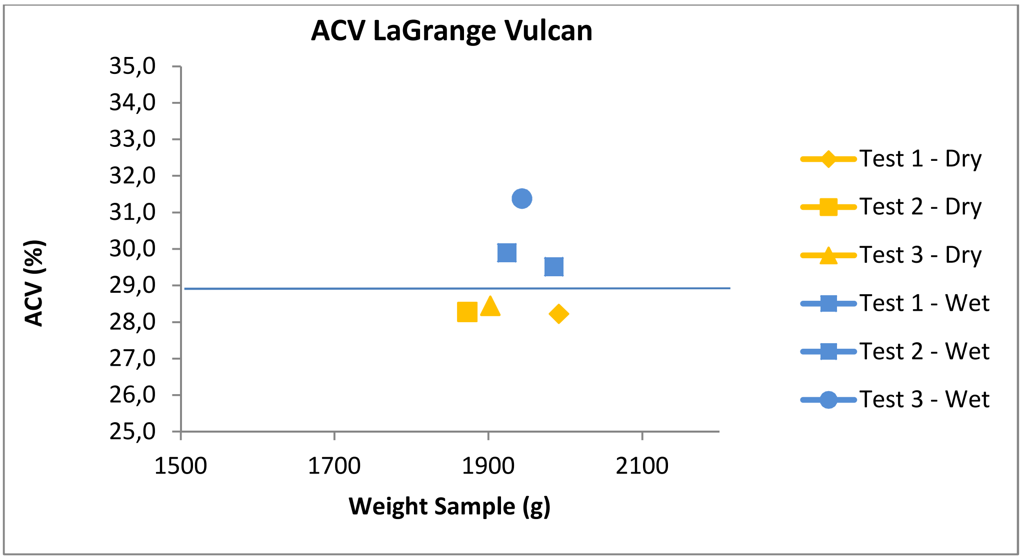


Figure 63 Distribution results of ACV test from LaGrange

As you can see from the table 44 and still better from Figure 63 which shows the maximum limit the ACV Wet results are beyond the limits prescribed by the regulations, and the ACV Dry results are very close to the limit.

11.3.2. Morgan county laboratory test

After having prepared the testers of granite taken from the Morgan County mine, was done, as the first test, the 10%FACT on three testers. In the Table 45 are showed the results of test and, among the notes, the deformations of testers at the end of test.

Table 45 10% FACT results of Morgan County

Sample	Weight material	Max Load	Passing at 2,36 mm	Note	10%FACT Dry Value	Limits min
N°	(g)	(kN/pound)	(g)		(kN)	(kN)
1	3003,4	338/76000	40,6	13mm,10 min	386	110
2	3020	430/96750	350	25mm,10 min		
3	3034,2	684/154000	1031,7	40mm,10 min		

10% FACT Wet Test: 75% of Dry test = 289,5kN

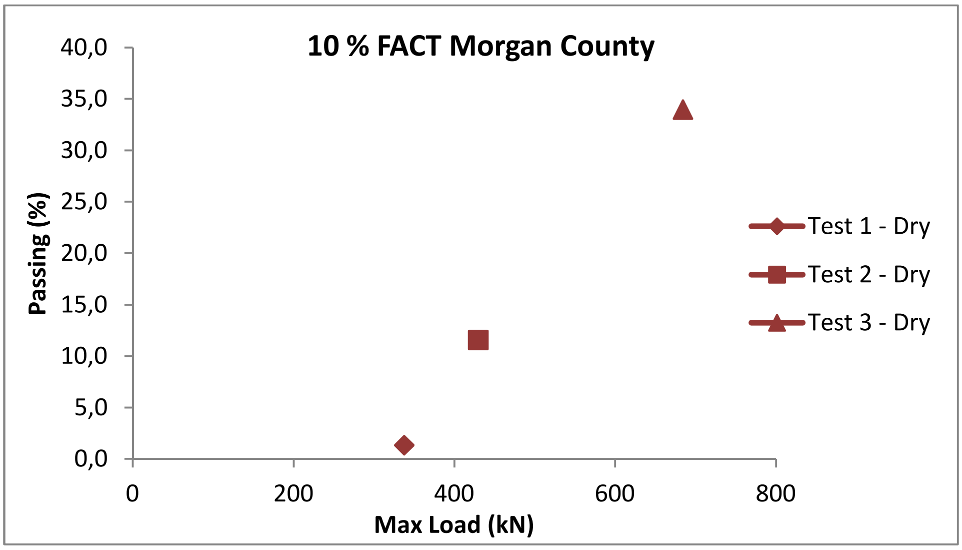


Figure 64 Distribution results of 10% FACT Morgan County

Then, was done the ACV dry test on three samples and later, the ACV wet test on other three samples. The results are showed in the Table 46 and in the Table 47.

Table 46 ACV dry results of Morgan County

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits max
N°	(g)	(kN/pound)	(g)	(%)	(%)
1	1802,2	400/90000	538,1	29,9	29
2	1764	400/90000	524,7	29,7	29
3	1824,4	400/90000	539,6	29,6	29

Table 47 ACV wet results of Morgan County

Sample	Weight material	Max Load	Passing at 3,35 mm	Value ACV	Limits max
N°	(g)	(kN/pound)	(g)	(%)	(%)
1	1930,8	400/90000	577,2	29,9	29
2	1847,7	400/90000	582,7	31,5	29
3	1817	400/90000	579,1	31,9	29

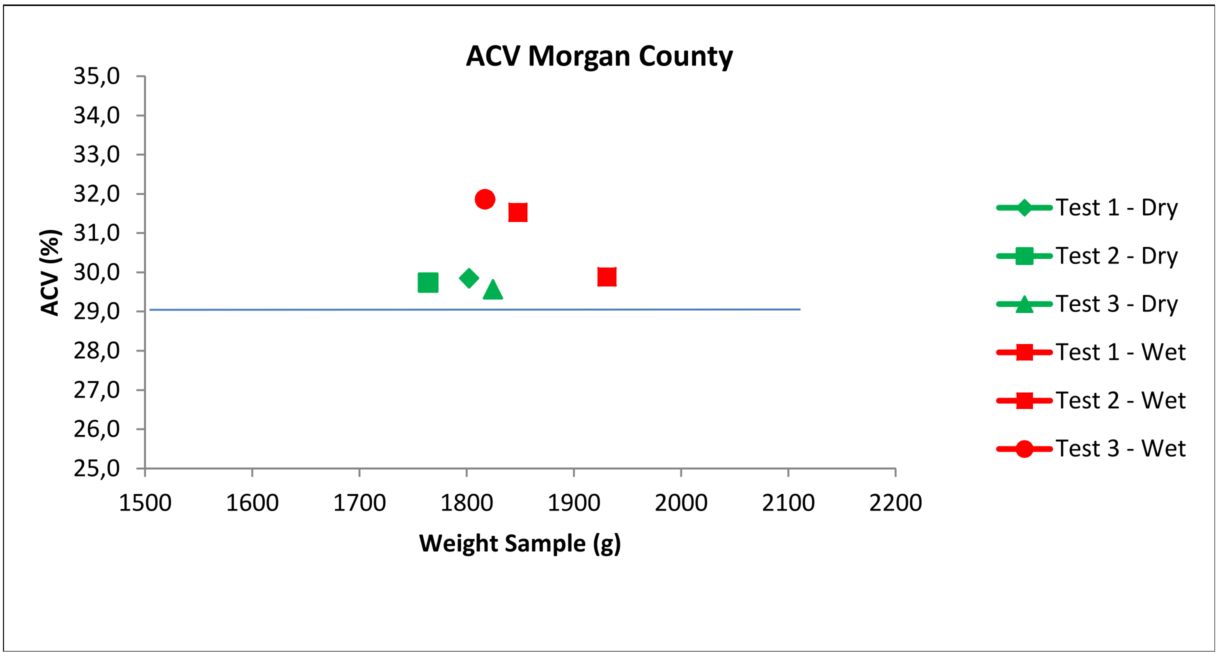


Figure 65 Distribution results of ACV from Morgan County

As you can see from the table 46, table 47 and from the figure 65, the ACV Dry and Wet results are beyond even if slightly the maximum limits prescribed by the regulations.

11.4. Some Discussion

In the laboratories of the Georgia Institute of Technology were done the main tests on the samples taken from two test sections with the purpose to better understand the mechanical features of the rock used. About the granite used in the LaGrange section, in the 10% FACT test, the average value obtained from three tests is 126 kN, higher than the minimum value for the achievement of test. In the ACV test this granite showed satisfying results in the Dry test and slightly higher than the value limit 29% in the Wet test.

About the samples of granite come from the Morgan County mine, the result obtained from the 10% FACT test showed results considerably higher both to the granite of LaGrange and to the value limit required by the Regulations. The ACV test, contrary to what we could think considering the results in the previous test, showed values higher than the limit required by the Regulations both for the Dry test and for that Wet.

In the following figures are compared the results obtained from two granites analyzed.

The Figure 66 concerning the results of the ACV test, you can see how the granite used in the LaGrange test section has a greater density and a greater mechanical resistance to crushing that, indeed, makes have a lower ACV value. The Figure 67 shows the results of the 10%FACT test from which you can understand how it is necessary much more energy for crushing the granite come from Morgan County compared to that used in LaGrange; a result that is opposed to that just reported in the Figure 66.

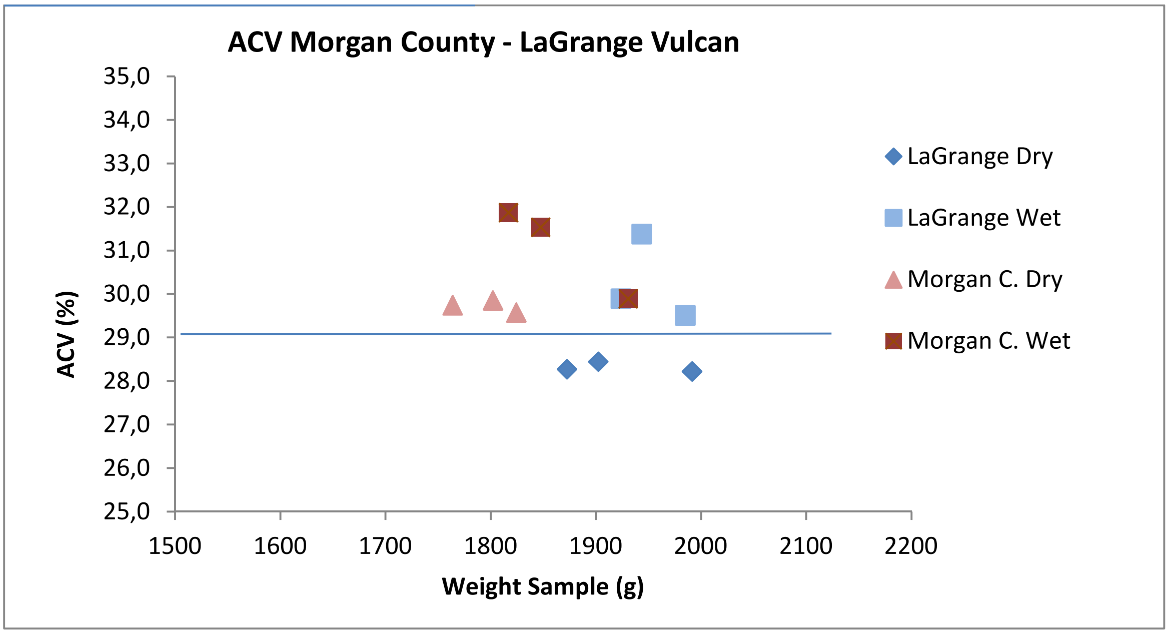


Figure 66 Comparison between ACV test results from LaGrange and Morgan County

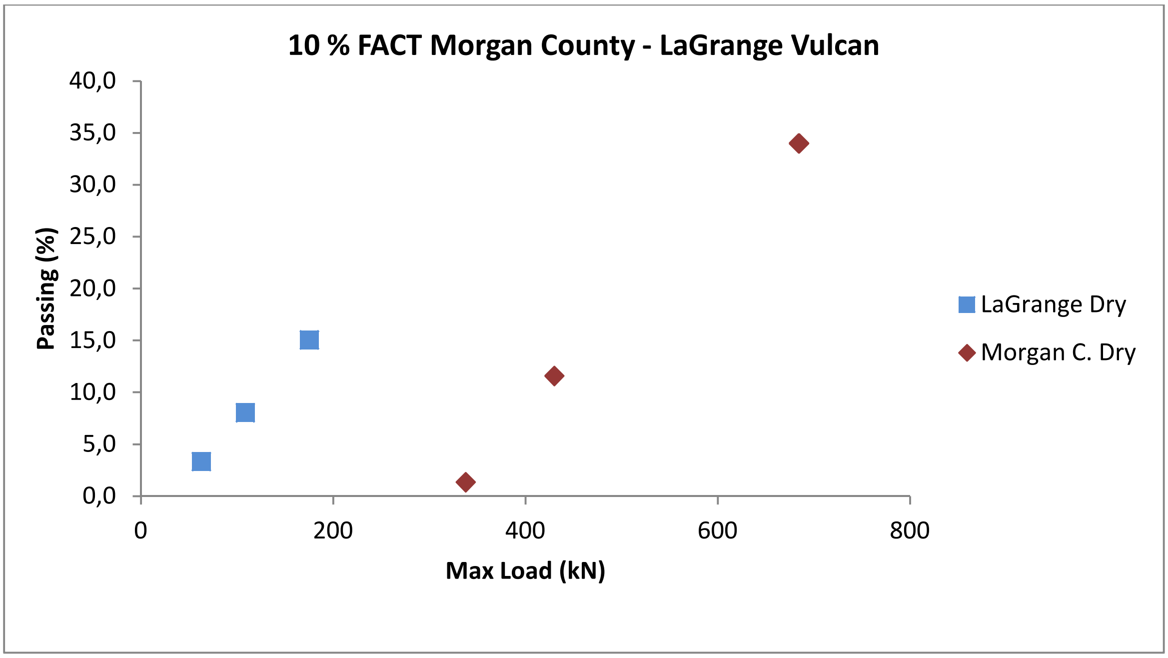


Figure 67 Comparison between 10%FACT test results from Lagrange and Morgan County

The Figure 67 shows simultaneously the results of the ACV test and of the 10%FACT test on both materials. In the figure triangle represents the average value obtained from the ACV tests on every materials, instead, square represents the average value obtained from the 10% FACT tests. From the analysis of the Figure 66 e Figure 67 emerge two opposing. That is given to the use of different grading in the ACV test and in the 10% FACT test and so it is possible to well understand how the mechanical reaction of rock depends on the grading itself and on the size of the minerals that constitute rock. More precisely, the granite used in LaGrange showed a mechanical resistance higher than the granite of Morgan County in the grading included between 19,0 mm and 13,2 mm that were used for the ACV test and lower in the grading included between 13,2mm and 9,5 mm that were used for the 10%FACT test.

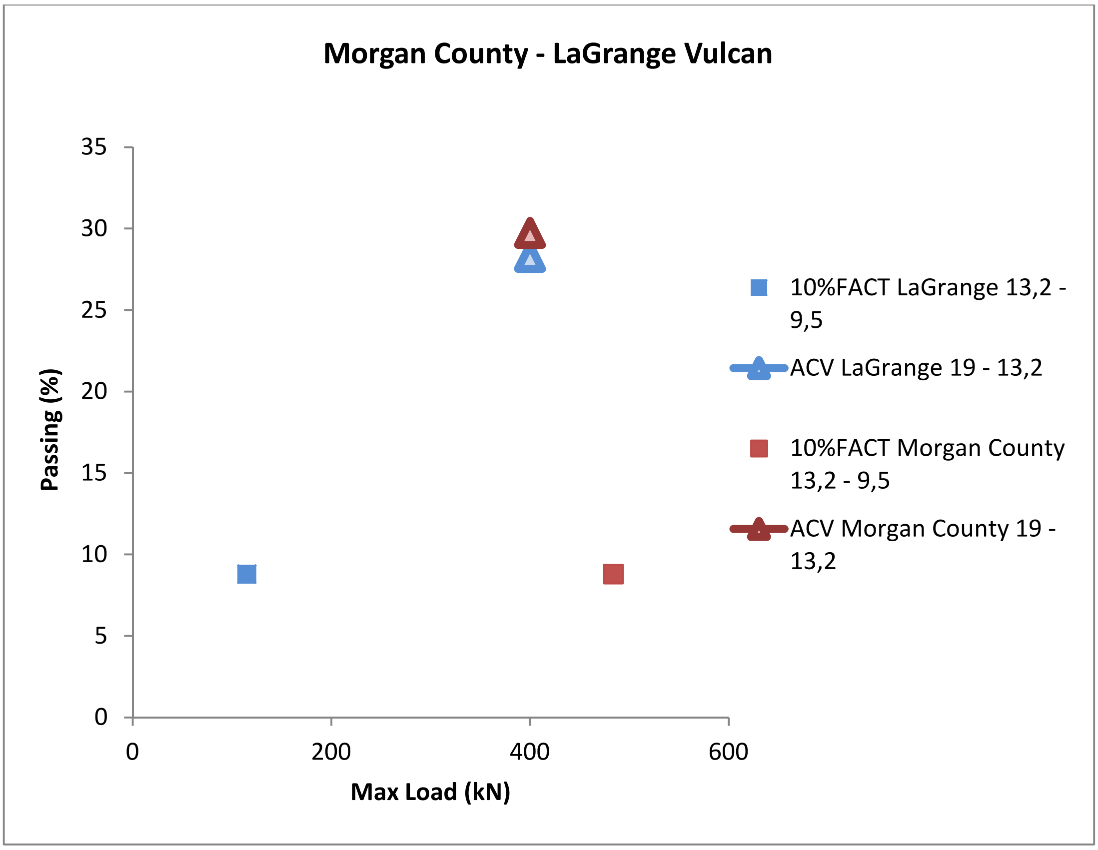


Figure 68 Comparison between ACV and 10%FACT results

12. CONCLUSIONS

Aggregates used in road design represent a mix of grains which has growing mechanical performance depending on the energy and compaction pressures. The need of creating stable and durable civil engineering works is closely related to the ability of compacting aggregates as more stringent possible. The development of compaction instruments (energy applied to the aggregates) has opened new and large perspectives that nowadays have been studying. Already since the '80s in South Africa technical regulations for the construction of infrastructures have considered the use of materials with high resistance to loads of static and dynamic type that allow to build paving with low economic costs and high environmental value, limiting the use of hot mix asphalt only to the top layer with restrained thickness.

The current economic situation has severely affected the Italian road infrastructure, and this affects both maintenance and future growth.

In this context, new alternatives with lower life-cycle costs would be welcome.

The Inverted Pavement Technique used in South Africa for more than 30 years can be a promising alternative.

The combination of the layers in the inverted base pavements can reduce the dependency of pavement construction on asphalt, while, at the same time, it allows a high quality pavement for all traffic levels by using the granular base as a structural key element.

In this study, we tried to establish if the materials currently mined in Sardinia could be used in the realization of Inverted Pavement, and in particular in the G1 base layer. This was done using, as a reference, the South African Regulations and the South African Dolerite as a material of reference. After the analysis of all the materials extracted, we dedicated ourselves only on Basalt, Andesite, Trachyte and Quarztite. After the first chemical analysis, it emerged that Basalt has a greater index of correlation with Dolerite, followed by Andesite, Trachyte and Quartzite.

These materials presented good Physical-mechanical properties except the Los Angeles value obtained by Quartzite.

The Atterbeg limits done proved that the Andesite analyzed cannot be used in Inverted Pavement, because of its plastic behavior, a feature that was found in the triaxial test too. The remaining mechanical tests considered by the Regulations and done on Basalt, Quartzite and Trachyte, 10%FACT and ACV, proved the good mechanical features of these materials. Through these tests, it was possible to obtain, specially for Quartzite, some results that follow perfectly the values limit required.

In the laboratories of the Georgia Institute of Technology were done the main ACV and 10%FACT tests on samples of Granite collected from the test sections with the aim of better understanding the mechanical features of the rock used. Notwithstanding the poor results obtained from the tests, the excellent condition of test section and the results reached during about 12 years, allow to prove how through materials different to those South African and through variations to the construction methods, the inverted base pavement structure is a promising alternative to achieve high quality roads at considerably lower costs than conventional pavements.

13. RECOMMENDATION FOR FURTHER RESEARCH

The results obtained through this work are a point of start for understanding if, in Inverted Pavement, materials different to those currently used can be utilized and for understanding how these can have an impact on durability and performances of the whole infrastructure. It is recommended that more tests should be performed in order to evaluate the behaviour of such materials in unbound road pavement layers.

A in-depth study could be done to find connections between type of aggregate and slushing process, to understand how material could affect slushing process, specially to study a methodology that allows to simulate the slushing process in laboratory.

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