ASSESSING THE ENGINEERING PROPERTIES AND PERFORMANCE OF LIMESTONE COMPARED TO CONTRACT SPECIFICATIONS AS ROAD CONSTRUCTION AGGREGATE IN THE KINGDOM OF TONGA

by

C Winston Hiliau

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Supervisor: Professor John Tookey

Department of Engineering

School of Engineering

AUT University of Auckland

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Attestation of Authorship

I hereby declare that this submission is my own work and that, to the best of my knowledge and belief, it contains no material previously published or written by another person (except where explicitly defined in the acknowledgements), nor material which to a substantial extent has been submitted for the award of any other degree or diploma of a university or other institutions of higher learning.

Signed: ……………………………………
C. Winston Hiliau

Date: 6 August 2018

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ABSTRACT

Flexible unbound aggregate pavement failure can be caused by a number of factors due to the nature and duration of repetitive loading being applied. These pavements are considered to be elastic multilayered system made up of a bound layer at the surface and unbound layers below. Modulus of elasticity, resilient modulus and Poisson ration properties of these layers are indicative of their capabilities to absorb and dissipate the compressive and tensional stress being subjected to minimize the load impact on the supporting subgrade material. Design objectives are therefore driven by ensuring the resultant strains are below the value of those that would cause deformation or damage to the pavements.

Aggregate material properties play a significant role in delivering the design objectives. More durable aggregate with less inclination to disintegrate under loading pressure are preferred to weaker material. This can be tested using a number of standard aggregate testing procedures.

The specific aggregate tests are usually included as part of the contract specifications so that the pavements are constructed with sound material reflecting the design requirements.

In this research, limestone aggregate samples were obtained from Tonga to be tested and assessed together with three historical case studies of project carried out in Tonga. Test results previously conducted by others were also reviewed.

Based on the analysis carried out, it was found that the contract specifications for unbound aggregate and chipseal projects in Tonga did not meet the minimum requirements for this type of work.

Further research is required to assess the possible use of reinforcement material such as fibres or stabilisation using cement and/or lime to strengthen the weak limestone aggregate. This could help alleviate further environmental degradation currently being caused by excessive limestone mining in Tonga.

Chapter One

INTRODUCTION TO THESIS

1.1 Background to thesis

Sustainable management of non-renewable resources has been a concern around the world for some decades. In recent times the issue of global warming, climate change and sea level rising has highlighted the critical need for sustainability. The principal challenge is identifying robust ecologically responsible and low-cost construction methods combined with sound materials in order to minimise or if possible eliminate environmental degradation.

This research intends to highlight an existing engineering issue that has become a major problem as the impact of western economic development pressures materialises in the Pacific region. The main focus is the engineering quality of limestone rock used as roading aggregate bituminous flexible pavement roads in Tonga's main island Tongatapu.

There are two main classifications of Tonga's road infrastructure which is based on the type of bituminous flexible surfacing. These are; main road or highways and trunk roads, refer Figure 1. Surfacing types used for Highways are asphaltic concrete (AC) and chipseal. Trunk roads are divided into two categories; chipsealed and unsealed surfaces. Unsealed surfaces are made up of coral and dirt roads. Tonga's 2012 Ministry of Infrastructure data showed 21 kilometres (kms) of AC and 287 of chipseal surfaced roads during information gathering for this work (2013-2014). No specific data on unsealed surfaces were available although these are illustrated in in Figure 1 or deduced from other infrastructure information.

It was predicted by McCotter *et al* (2010) that between 2010 and 2014, 63,255 tonnes of limestone aggregate to be used for six different road and airport infrastructure resealing projects.

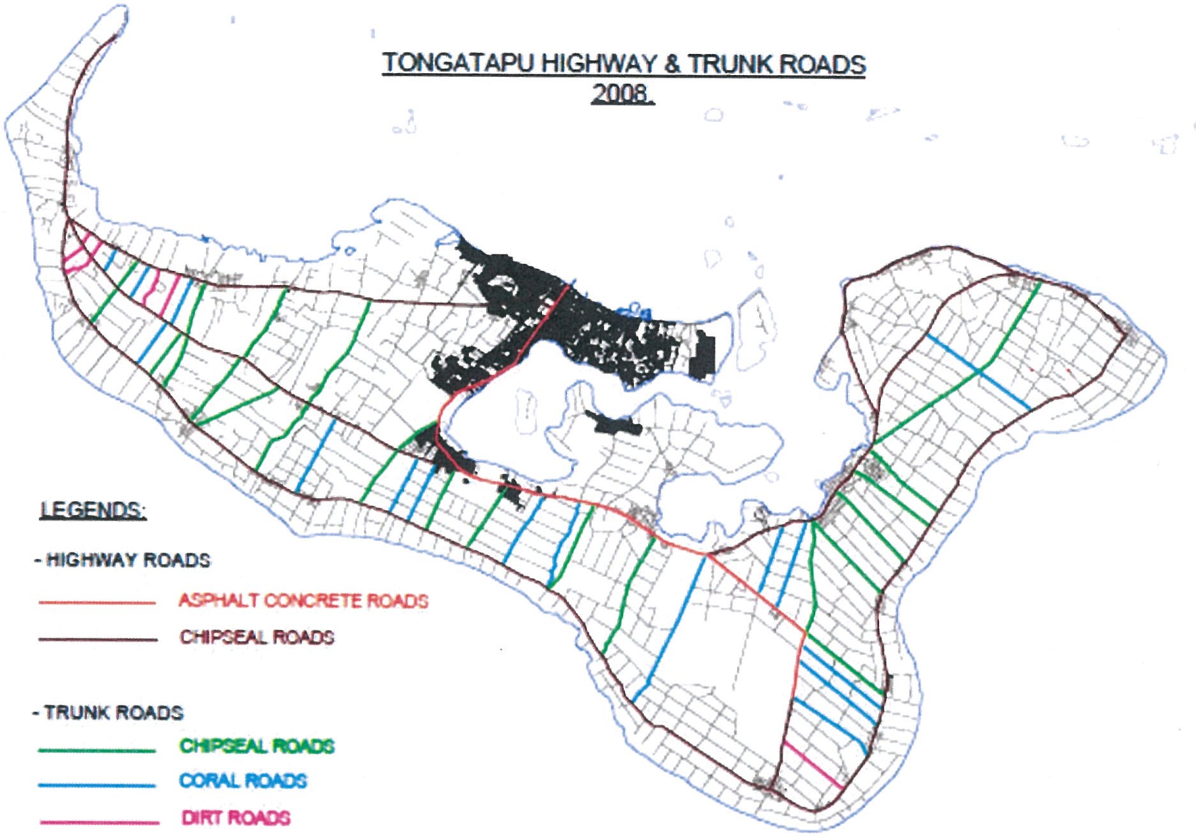


Figure 1.1. The Highway and Trunk Roads of Tongatapu the main island of the Tonga Islands (Ministry of Infrastructure (MOW), Tonga 2012).

1.2 Problem Statement

The current state, performance and deterioration rate of Tonga's roading infrastructure impacts significantly on the country's future development mirrored in the wider Pacific region. New Zealand and Australia have been funding via financial aid numerous Pacific states economic development including Tonga but not much focus on fundamental technical infrastructure problems that have persisted for decades such the limestone aggregate.

Tonga's limestone is marine formed, a renewable resource, and the only material available for road construction in Tonga. However, for the purposes of this work, it is considered non-renewable due to negligible replenish rate. The current extraction rate far exceeds the replenish rate and only a limited volume is accessible for various reasons.

Despite the millions of aid dollars invested on construction and reconstruction of road infrastructure in last 25 years, majority of new pavements continue to deteriorate much at a faster rate than standard design life expectations of similar flexible pavements, which depending on design and material used can be 15+ years.

Majority of aid are granted with an attachment of technical expertise and nominated contractors from the donor countries therefore design, contract specifications and management are controlled by external experts. These are to be discussed in relation to aggregate requirements.

The rate of demand on non-renewable resources such as limestone aggregate is a prime example of worldwide 'non-renewable resources' requiring careful management in order to sustain future demand. Sustainability is a challenging issue and it is a very common theme throughout the South Pacific region.

1.3 Objectives and scope of thesis

The first objective of this thesis is to assess limestone aggregate properties and compare with appropriate specified requirements for road construction projects.

The selected aggregate values to be compared are:

- Particle size distribution,
- Crushing resistance and

- Weathering quality.

The results are also compared and discussed in combination with tests carried out by others, focusing on application and data significance.

The combined test results are critiqued then compared with historical road construction project specifications from Tonga.

New Zealand Transport Authority (NZTA) aggregate pavement specifications for unbound aggregate flexible pavement construction will form the foundation of data appraisal. The standard requirements for basecourse TNZ M/4, sealing chip TNZ M6 and asphaltic concrete TNZ M/10 will also be reviewed and compared to the tested limestone properties.

To achieve this, aggregate samples were extracted from Tonga quarries (Chapter 2) and tested at OPUS International Laboratory in Auckland. The volume of aggregate used and the number of tests were restricted due to funding and quarry access issues.

The second objective for this research is to highlight the environmental impact issues generated as a direct result of not addressing the technical deficiencies associated with limestone. Continuous and uncontrolled extraction of limestone is causing substantial environmental degradation not only to the scarcely available land but it is also a major threat to Tonga's vulnerable underground water supply.

Test results and environmental issues highlighted in this thesis will provide a basis for further research possibilities and development.

1.4 Justification for the research

The introduction of National Strategic Planning Framework (NSPF) in 2010 and the National Infrastructure Investment Plan (NIIP) in 2012 to improve Tonga infrastructure were important steps forward led by the Government, but both were literally a plan and in principle only. They did not contain specific references on to address the fundamental issues relating to the causes of continuing roading infrastructure problems. However, addressing the road infrastructure problems was an essential requirement in order to achieve objectives referred below,

1.4.1 Tonga Government's NSPF and NIIP

In a sustained effort to improve social and economic development by reprioritising national infrastructure planning, the Tongan government compiled a five to ten year framework, NSPF 2009/2010 – 2014/2015 and the NIIP 2010). The government's main vision was: "to create a society in which all Tongans enjoy higher living standards and better quality of life through good governance, equitable and environmentally private sector-led economic growth, improved education and health standards and cultural development" (NIIP, 2010, p. 2).

The four main objectives were essentially to improve quality of life, access to business, sustainability of renewable energy, resource management and maintenance (NIIP, 2010).

Several factors were identified as the main drivers for improving infrastructure, summarized in Figure 1.2.

The NIIP (2010) document highlights priorities and plans for infrastructure initiatives in energy, telecommunications, potable water, waste-water and transport which included 880 km of roads, including community roads, 40% of which are sealed.

The overall direction and priorities of national infrastructure planning and the NIIP are shaped by the National Strategic Planning Framework 2009/2010 – 2014/2015 (NSPF).

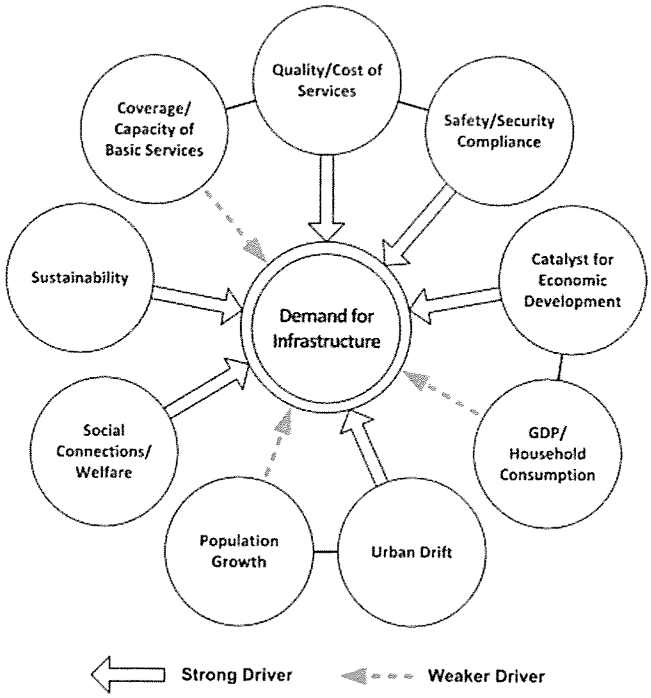


Figure 1.2. Infrastructure demand ***"drivers"*** for Tonga (from NIIP, 2010)
The NIIP 2010 sets out the country's priorities and infrastructure initiatives up to 2015. This is the working document for the government whereby progress can be monitored and updated.

The Tongan government's aim is to establish a robust national infrastructure which is well planned and maintained as a high-priority goal.

"Appropriately well planned and maintained infrastructure that improves the everyday lives of the people and lowers the cost of business – by adequate funding and implementation of the National Infrastructure Investment Plan (NIIP) (Government of Tonga 2012a). Continuing progress to a more efficient government by focusing on its core functions, improving coordination, service delivery and optimizing use of resources. (Government of Tonga, 2012b)

The demand for better road infrastructure is driven by population growth and company or individual's economic activities, striving to achieve good quality of life. Identifying an important problem associated with having reliable and good quality roads is an important objective of this research.

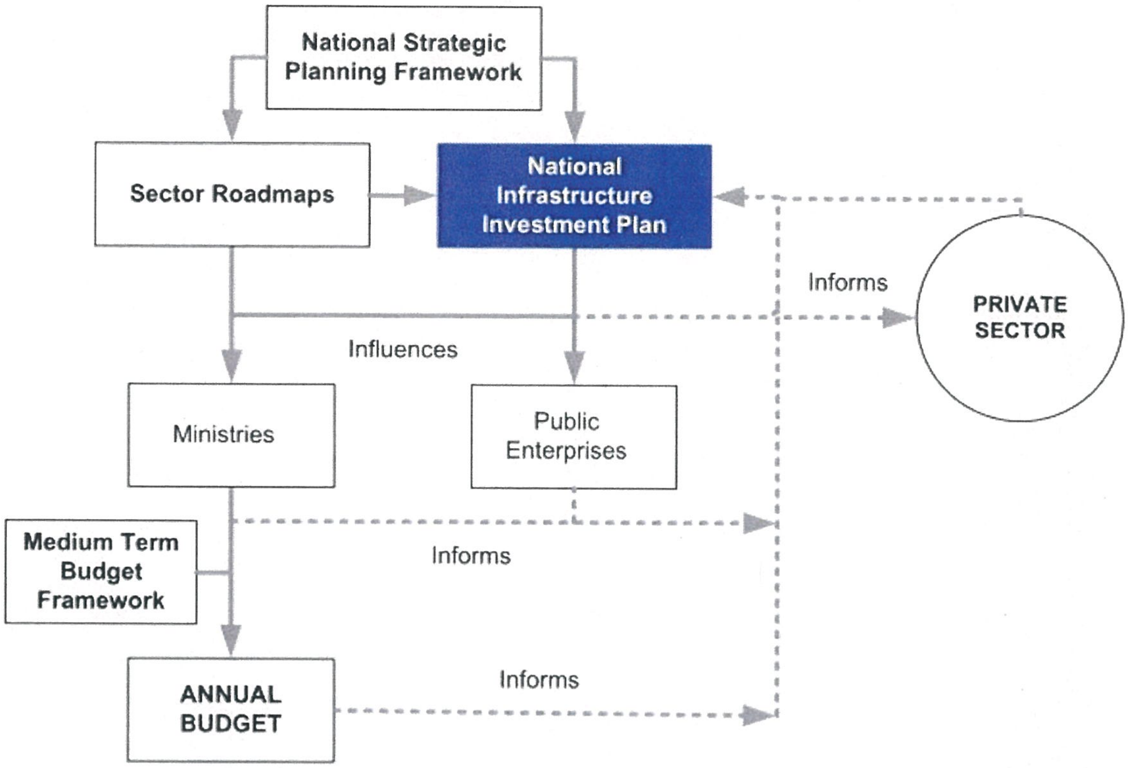


Figure 1.3. Illustrates the relationship between NSPR and NIIP (NIIP, 2010) NSPF.

Figure 1.3 shows the flow on effect allowing for Ministries and Public Enterprises investment planning aligning with Government Medium Term Budget Framework. In addition, the Private Sector is also included so that flow on information is channelled through allowing for decision making on future planning.

1.5 Research Methodology

The aggregate testing work was carried out for ***objective one*** in following order:

1. Review of the geological nature of the limestone aggregate and carry out site visit to obtain samples for testing.

2. Determine the Particle Size Distribution envelope for Tonga's limestone - NZS 4407: 1991, Test 3.8.1 Wet Sieving Test.

3. Determine the crushing resistance of aggregate samples from existing sources - TNZ 4407: Test 3.10

4. Determine Weathering quality - TNZ 4407: Test 3.11

5. Review the current or past contract specifications for road construction projects in Tonga.

6. Compare the test results with the contract specifications identified in above.

7. Determine if the test results has any similarity to any of the New Zealand Transport Agency's TNZ M/4 Regional Basecourse properties. Discuss.

Objective two is to stress the environmental impacts and the long term damage likely to be caused due to the uncontrollable extraction of non-renewable resources such as limestone in Tonga. The issues to be addressed are not exclusively Tonga's but is applicable to all Pacific island countries and all countries with similar limitations of non-renewable resources.

Therefore the essential questions to address are:

- Does the locally sourced limestone meet the standard contract specifications for road construction projects or not?
- How does the continuous limestone demand impacts on non-renewable resource?
- What are the future sustainable options?

Success in developing an understanding of local material performance as well as developing new solutions to prolong pavement life will have a significant impact in Tonga the Pacific. Better knowledge and command of pavement design performance would allow for more effective planning of scheduled maintenance, which commensurately increases the probability of infrastructure achieving optimal life expectancy.

1.6 Organsation of thesis

The thesis is structured into 7 chapters progressing from Tonga's geology, pavement design principles, relevant pavement design specifications, to selection and testing of aggregates.

Chapter 1 provides the background resulting in the need for this work and introduction highlighting the objectives of the research. The demand on the locally sourced limestone and local versus international sustainability management are introduced.

Tonga's two crucial frameworks that form the basis for redevelopment, the National Strategic Planning Framework (NSPF) 2009/2010 and the National Infrastructure Investment Plan (NIIP) 2010 are introduced. Methodology and organisation of this work is briefly covered.

Chapter 2 introduces location and geology background to the origins of Tonga's limestone aggregate. Properties of the limestone are discussed as well as the location of quarry sites. The relevant literature is discussed.

Chapter 3 has two parts. The first reviews the principles of pavement design and construction application in relation to aggregate requirements using New Zealand and Australian standards as benchmark. Introduction of Flexible Pavement Design concepts are discussed, pavement loading and the basic aggregate properties needed to provide structural integrity. The relationship between pavement design and construction is appraised.

Reviewing empirical pavement design method which applicable to Tonga. Aggregate performance under loading is discussed particularly relating to imposed stresses and strains.

In ***Chapter 4***, Population growth in Tonga's mainland Tongatapu, population density and transport demand are discussed.

Tongan roading infrastructure is introduced and reviewed in some detail compared to theoretical design concept and philosophy.

Chapter 5 is a case study of three historical contracts are evaluated highlighting the specified aggregate selection, testing and approval processes. How relevant were the quality assurance specifications to Tonga.

Also included in this chapter will be references to New Zealand Transport Authority (NZTA) and AUSTROADS standards that were either specified or deemed relevant for comparison purposes.

Chapter 6 covers aggregate testing principles and methods. How do test results inform design?

The process of aggregate sampling, testing carried out for this work is discussed. Testing done by others will also be reviewed and discussed.

Majority of Tonga's road infrastructure surfacing comprised of chipsealing hence requirements for chipsealing aggregate will be referred here.

Chapter *7* concludes the thesis with the summary of the research findings. Some proposals will be provided for future research with the objective of raising possibilities to address the findings.

Correlations between reviewed test results and limestone aggregates ability to deliver the intended outcome is finalised.

Research limitations will be highlighted to identify possible future work.

Chapter 2

REVIEW OF GEOLOGICAL BACKGROUND, LIMESTONE, AND QUARRY SITES

This chapter introduces location and geology background to the origins of Tonga's limestone aggregate focusing only on relevant information to the mainland of Tongatapu. Literature review of limestone properties as well as highlighting quarry sites.

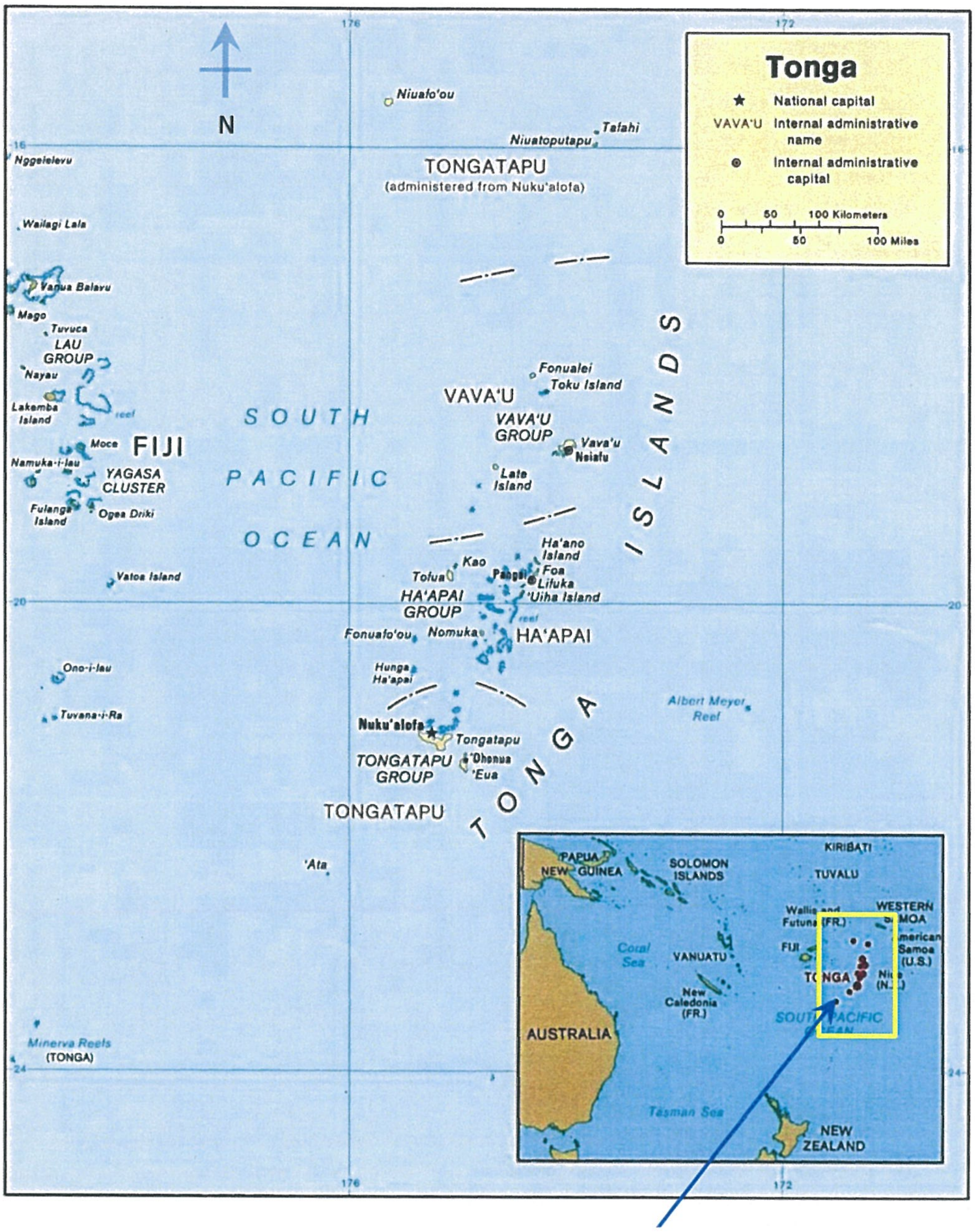
One of the challenges faced during this work was the lack of up to date geological studies of Tonga's geology. Engineering studies are also limited reflected in the sporadic available literatures.

2.1 Geological background

Tonga is situated at the eastern most edge of the Australia-India Plate formed as a response to the subduction of the Pacific Plate beneath the Australian-Tonga Plate as stated by Francheteau (1983). Refer to Figures 2.1 and 2.2.

It is an archipelago in the south pacific located to the east of Fiji and south of Samoa. There are three main island groups Vava'u, Ha'apai and Tongatapu out of 176 islands, 36 of which are inhabited including the main islands. Tongatapu is the mainland where the capital Nuku'alofa is located and the most populated, 75% of its 103 036 population resides here, highlighted in the 2011 census (Tongan Govt. Census, 2011).

Pelesikoti (2003) referred to the Constitution of Tonga in 1887 declared Tonga's boundaries as being between Longitudes 177° - 173° W, and latitudes 15° - 23° 30'S (see Figure 2.1 below). Pelesikoti (2003) also described total area of the Exclusive Economic Zone (EEZ) of Tonga had increased to approximately 700,000 km^2 compared to the 397,282 km^2 covered by an 1887 Royal Proclamation.



Tonga Islands

Figure. 2.1 Map of the south Pacific Islands (insert) and Tonga Islands stretching along a distance of 800km on north – south line. (https://en.wikipedia.org/wiki/File:Tonga.jpg)

2.2 Geological formation of Tonga

The Tonga islands were formed as a result of collision and the subsequent underthrusting of the Pacific Plate beneath the Indo-Australian plate, in mid Eocene (45 Ma). This development is still ongoing and has resulted in the formation of a complex pattern of subduction, island arc volcanism, backarc basin formation, and rifting.

The subduction of the Pacific Plate underneath the Australia-Indian Plate has resulted in the formation of two large submarine ridges at the edge of the overlying Indo-Australian plate, that are aligned parallel to each other in a north-north-east (NNE) direction (Figure 2.2 and 2.3).

The ridge to the west of Tonga contains volcanic islands and a center of frequent submarine volcanic activities. The eastern ridge has no active volcanoes but contains the three main coral islands of the Tonga Group. From south to north they are: the main island group of Tongatapu, about 150kms to the north is the island group of Ha'apai, a similar distance further north is the island group of Vava'u, and further north still towards Samoa are the islands of the Niua groups. To the east of the ridges, running parallel NNE, is the Tonga Trench, which is the second deepest trench in the world behind the Marianas Trench.

Table 2.1 shows a summary of the island groups and some of the physiographic features.

Island Grouping	Main Islands in the Group	Capital Towns	No. of islands in the Group	Total land area of the Group (sq km)	General Physiographic Features
Tongatapu Group	Tongatapu	Nuku'alofa	17	347.92	It is low relief, rises from broad tidal flats and lowland coastal swamps and mangrove forests along the northern edge to a maximum elevation 85 metres towards the south east. Most of Tongatapu is less than 17 metres above mean sea level. The upwind south coast has rugged limestone cliffs and terraces from 6 to 46 metres in height. A narrow fringing reef surrounds the east, south and west coast of the island. Coral reefs extend northward from the north coast line along an extensive submerged terrace. There is an extensive shallow lagoon in the central part of the island (Fanga'uta and Fangakakau). It extends 6-10 km into the island and is surrounded by low-lying swampy areas.
	‘Eua	‘Ohonua			One of the oldest (geological) island in Tonga, high relief, minimum relief of 312 metres, no lagoons, few caves and fresh water springs.
Vava'u Group	Vava'u	Neiafu	97	192.69	Vava'u has distinct terraces to the highest point at 213 meters, extensive tidal flats, and fringing reefs, submerged barrier reef on the eastern side of the group, protected harbours, well developed beaches, and numerous small islands.
	Niuas	Angaha			Volcanic, 260 m above sea level, small internal lake in old crater in the middle of the island, barrier reef, active volcano at Niuafo'ou.
Ha'apai Group	Lifuka	Pangai	62	109.30	Low relief generally only a few meters above mean sea level, exceptions the volcanic islands of Kao (1,030 m asl) and Tofua (507 m asl), numerous small islands formed on coral reef platforms, extensive coral reefs.

Note: Descriptions of major physiographic features are also included. Tongatapu is the largest island and the capital Nuku'alofa is located here. The number of islands listed in the No. of islands in the Group includes some that are uninhabited (Pelesikoti, 2003).

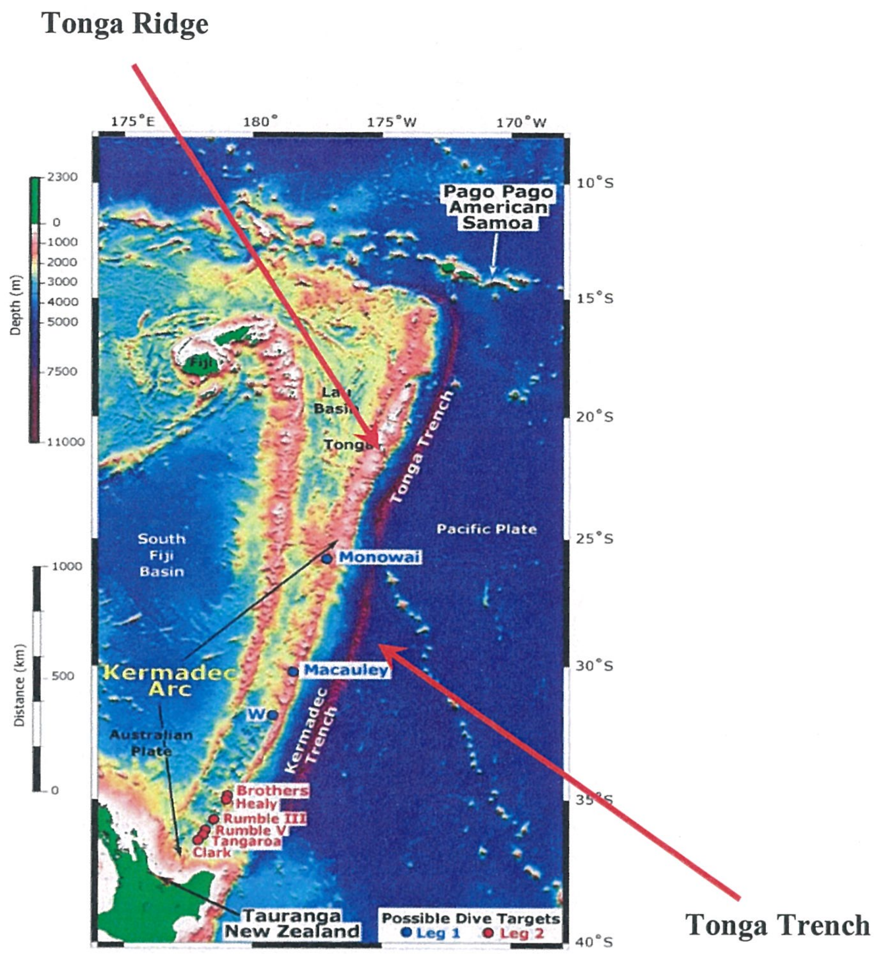


Figure. 2.2. Map illustrating the location of the Kermadec Trench running north-east from the New Zealand's North Island towards Tonga. Sources: Figure 2.2 (wikimedia.org/wikipedia/commons/d/d4/Kermadec_Arc.jpg)

Tongatapu the mainland is located at the southern end of the island group and the two Niuas are located at the most northern end.

Figure 2.2 shows the Tonga Islands located on top of the Ridge which lies to the west of the Tongan Trench. The subduction zone where the Pacific Plate is pushed below the Indo-Australian Plate.

There are five separate fault blocks on the Tonga Ridge identified by Cunningham & Anscombe (1985). The surfaces of these blocks are less than 200m deep with irregular limestone caps. Parts of these caps protrude out of the ocean which forms the islands of Tonga

(Roy, 1990; Cunningham & Anscombe, 1985). Tongatapu and 'Eua islands are on the southern block end and there's a lagoon to the north with depth of up to 1000m (Figure 2.3).

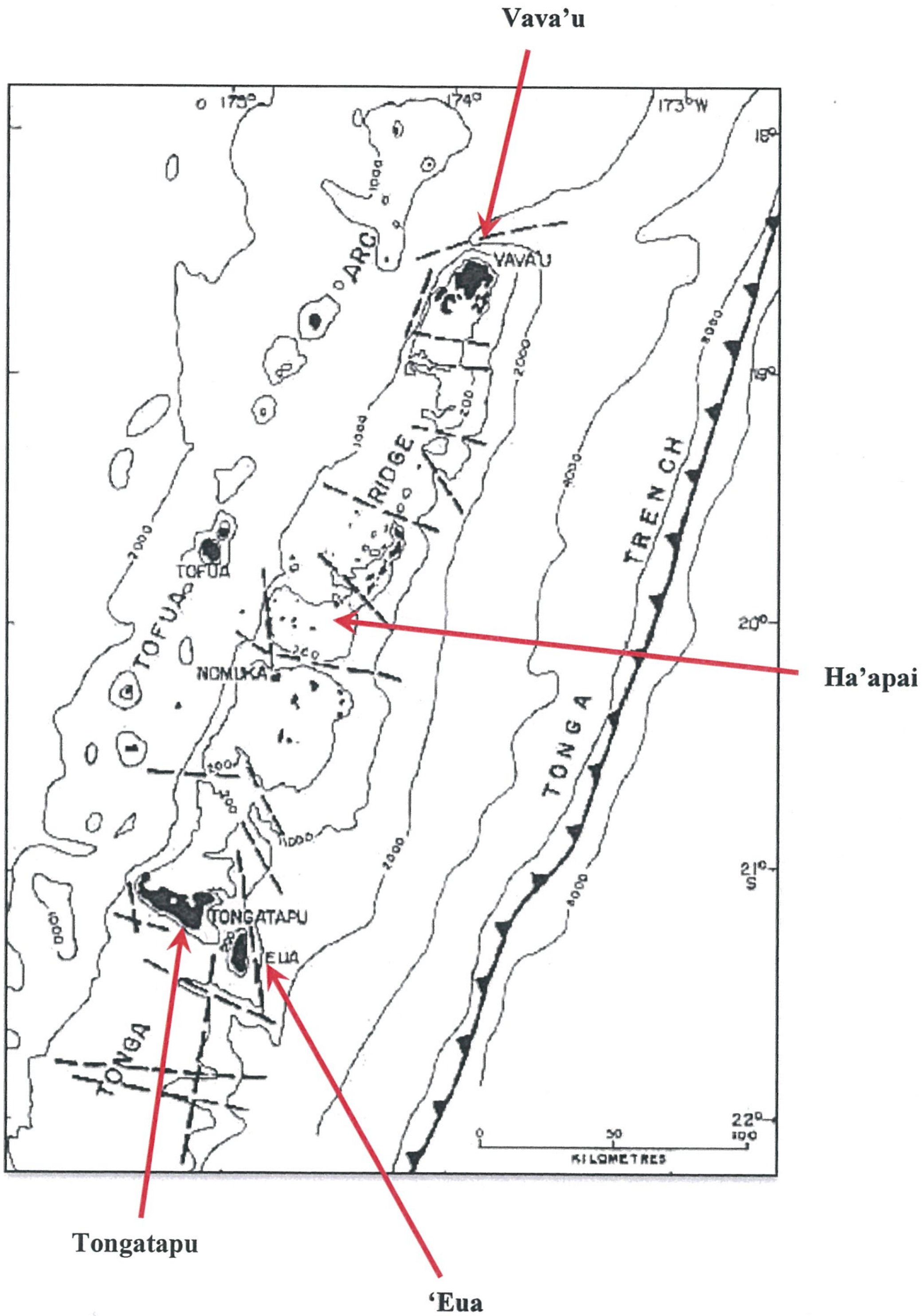


Figure. 2.3. This map illustrates the location of the Tonga Trench in relation to the Tonga Ridge. The dashed lines show some of the fault lines at the Tonga Ridge (modified from Roy, 1990).

Note: The contour depth is in metres and a short distance from the ridge to the one of the deepest trenches in the world

It's the opposite at the Vava'u block, top of the Tongan Ridge north of the Ha'apai Group. Vava'u islands are to the northern and it is tilted to the south as identified by Roy (1990) & Taylor (1978).

The Tonga Trench, located to the east of the Tonga Island Group (Figure 2.3), "the second deepest ocean trench (> 10, 800m) in the world, lies parallel to the east of these submerged ridges (Pelesikoti, 2003; Scholl. Et al., 1985: Cunningham & Anscombe, 1985).

Tonga's geological formation was divided into three main groups by Taylor & Bloom (1987) highlighted by Pelesikoti (2003). They are:

- Uplifted limestone from submerged sea floor caused by the subduction plate, such as Tongatapu, Vava'u and Ha'apai;
- Volcanic islands such as 'Ata, Tofua, Late, Niuafo'ou and Tafahi; and
- Mixed limestone and volcanic islands such as 'Eua and Niuatoputapu

2.3 Regional Geology

All Tongan Islands have well-developed soils rich in volcanic material (Roy, 1990).Roy (1990) confirmed that limestone is up to 250m thick in places that can be located on Tonga's three main island groups. These three islands "are elevated masses of tertiary limestone and volcanic capped quaternary limestone, which rises above a central platform" (Pelesikoti, 2003). These are thickly covered by soils derived from weathered volcanic ash up to 3m in depth and there are no significant igneous extrusions near the surface except for minor exposures on the island of 'Eua.

Tongatapu's soil has been described as andesitic volcanic ash and classified as Mollisols based on the U.S Department of Agriculture soil classification. These soils have been around for 5 to 20,000, years blown originally over as volcanic ash from Kao and Tofua to the northwest of Tongatapu on the Tofua Ridge (Figure 2.3).

The more elevated limestone, south of Tongatapu, derived from palaeoreef corals are relatively massive, cream coloured biomicrites and biosparites. There are numerous cavities, but they are often well cemented. Patches of less dense friable material occur, but they are not widespread. In some deposits there is much secondary growth of calcite cement

(Harrison, 2003). The limestone tends to be softer, more porous and of lower density moving towards the north (Figure. 2.4 (a) and (b)).

(a) (b)

Figure. 2.4 (a) 'Ahononou Quarry in the south (Harrison, 1993) and Figure 2.4 (b) Pili Quarry to the north in Nuku'alofa (Author's site visit 2012)

2.4 Morphology

This work focuses on the main island Tongatapu however geological morphology similarities to the other islands is also discussion providing an overall understanding of the limestone rock origins.

As referred above, the subduction zone to the east of Tonga is responsible for the uplifting the edge of the Indo-Australian Plate by the Pacific Plate (Pelesikoti, 2003). The age and makeup of Tonga's limestone is common knowledge to geologists that have carried out fieldwork in the past such as Cunningham and Anscombe (1985).

It's estimated by Roy (1990) that 130 – 250m limestone capping Tonga developed between 11,700 and 5.33 million years ago, the Pliocene and Pleistocene epoch, . These were raised through subduction zone activities forming the cliffs at the southern end of Tongatapu. The cliffs are relatively narrow and irregular, 0.5 – 1.25 km wide, rising to 20m above sea level from the northwest, the southern tip and to the northeast (Roy, 1990). Refer points A, B & C respectively in Figure 2.5 (a).

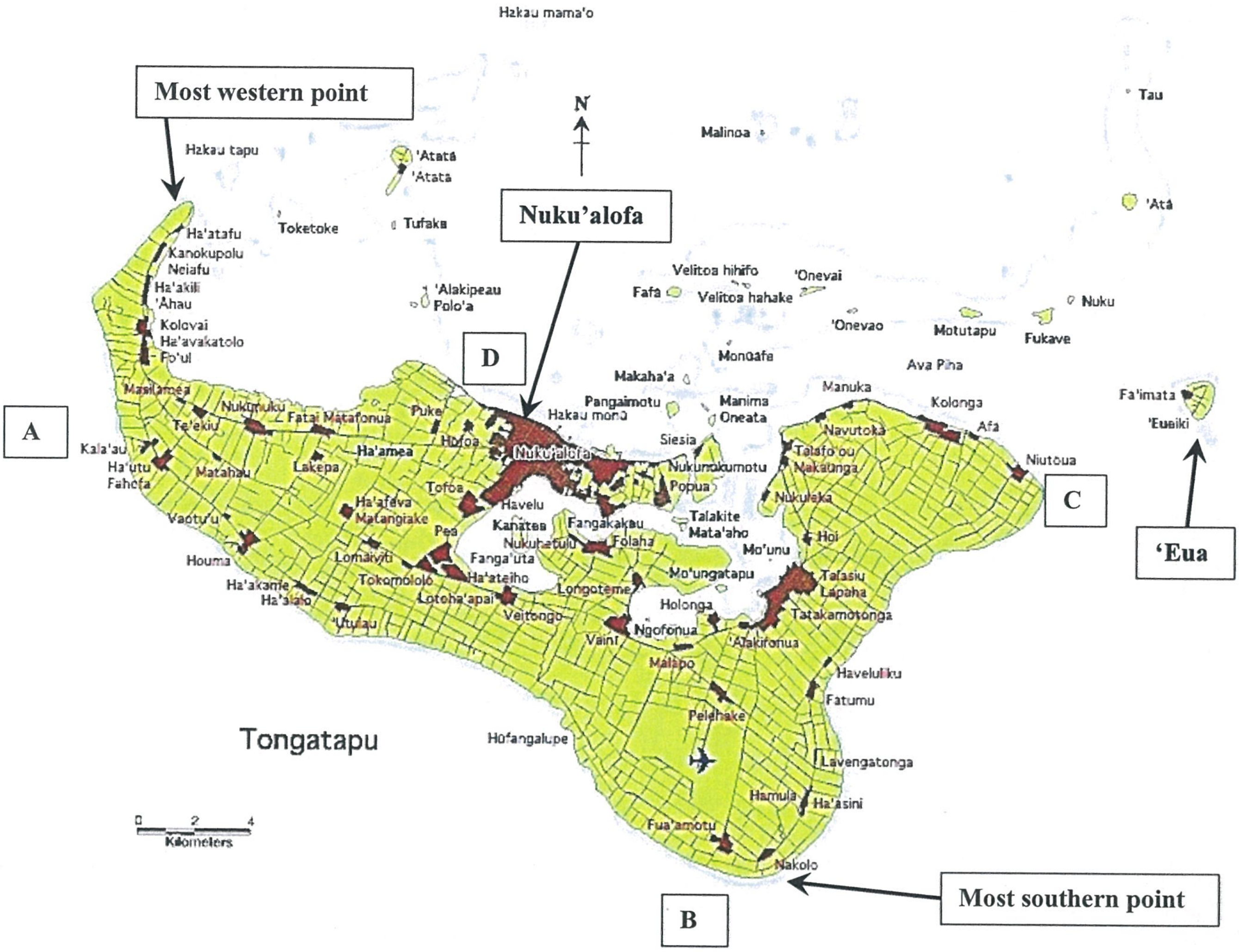


Figure. 2.5. Tonga's main island Tongatapu illustrating the various villages (Source: Modified version from Wikipedia 2015 https://en.wikipedia.org/wiki/File:Tongatapu.gif).

The island is raised at the southern coastline from point A, to the most southern and highest at the bottom point B and to point C to the south east. It then slopes down toward the capital Nuku'alofa point D.

The "crescent-shaped ridge" (Harrison 1993), points A, B and C in Figures 2.5 and 2.6, has been referred to by Perring and Mansergh (1989) as the oldest palaeoreef structure on Tongatapu. It is relatively flat to the north apart from some occasional steep sided hill of up 20m high.

These colossal palaeoreef limestones are biomicrites[1] and biosparites[2] cream in colour (refer Figures 2.4 (a) & (b), 2.7 (a) (b) (c) & (d)).

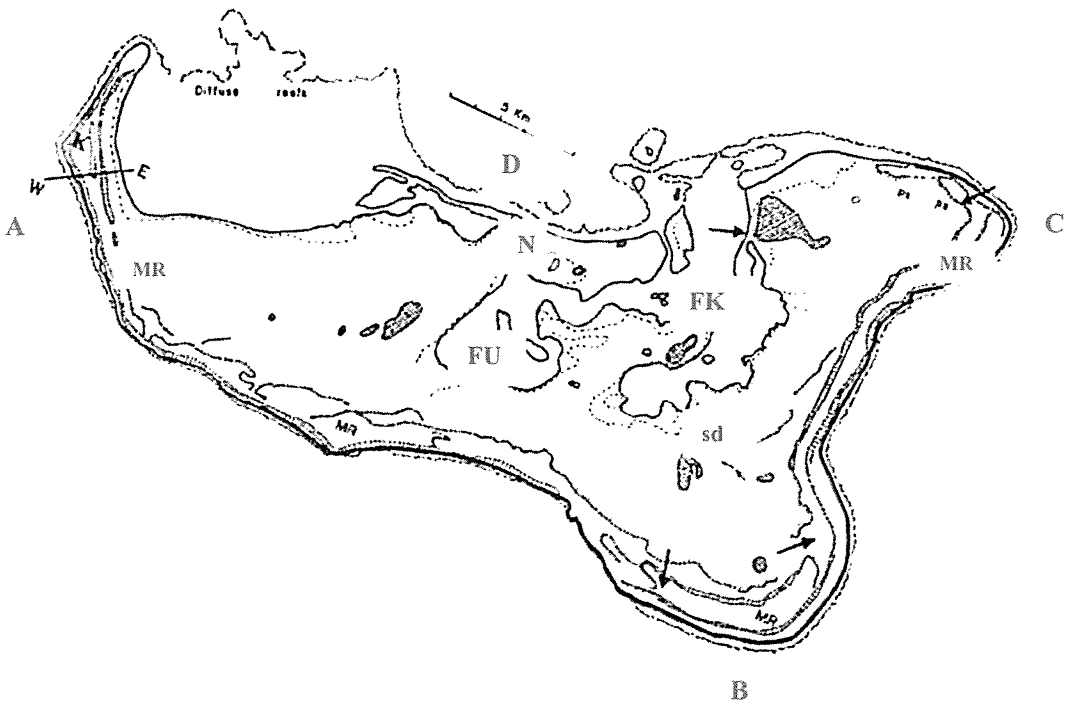


Figure. 2.6. Highlighted in darker bold lines the cliff faces referred to in Figure 2.5 (a). FU = Fanga'uta, FK = Fangakalau, N = Nuku'alofa, K = Kolovai ridges (Modified map from Roy (1990)).

The condition of the limestone is "well cemented" by Harrison (1993) considering that there are many cavities and vughs present due to corals and algal-bound masses found within the rocks.

There's evidence of patch reefs located to the north of the ridge line in the lagoonal areas where the limestone is less massive, less cemented and are generally low density. It's more of a mix between coral/algal bound limestones and hard limestones with some partially consolidated detritus.

[1] Biomicrites. A limestone consisting of bioclasts set in a micrite matrix. It is the product of a poorly sorted accumulation of shell fragments and mud (Biomicrites, n.d.).

[2] Biosparites. A limestone consisting of bioclasts together with a sparry calcite cement (sparite). It is the product of an accumulation of clean-washed, mud-free shell debris, with diagenetic cement growth (see diagenesis) in pore spaces. See folk limestone classification (Biosparites, n.d.).

Harrison (1993) further described the lagoonal deposits as "generally not exposed as they are usually concealed beneath the thick soil. They are however composed of soft, porous, friable and rubbly limestones with some algal bound limestones". Harrison (1993) also found calcite cement secondary growth as well as some friable and less dense detrital[1] limestones.

The following photos are from the southern coast of Tongatapu above, refer Figure. 2.6 marked B, illustrating height above sea level of the seaside cliffs and weathered limestone rock faces (Source: www.panoramio.com, 2015).

Figure. 2.7 (a)

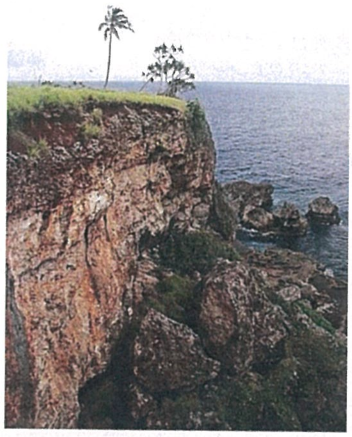


Figure. 2.7 (b)

Figure. 2.7 (c)

Figure. 2.7 (d)

Figure. 2.7 (a), (b), (c) and (d) Cliffs around the southern coast of Tongatapu (Extracted from www.panoramio.com, 2015).

[1] Detrital. Detrital particles can consist of lithic fragments (particles of recognisable rock), or of monomineralic fragments (mineral grains). These particles are often transported through sedimentary processes into depositional systems such as riverbeds, lakes or the ocean, forming sedimentary successions (Detrital, n.d.).

Tongatapu's morphology outside the cliffs identified is relatively uneventful and the "land surface is flat to gently undulating with occasional steep-sided hills 10-25 m high and linear, scarp-like features" (Roy, 1990).

Hills are randomly distributed ranging from some hundred metres in diameter to 1 km. Gentle scarps with grades variation between 1:20 to 1:100 were mapped by Roy (1990).
These scarps appear in two settings: They are:

(i) Scarps and slope zones delineate both sides of the main ridge. Except for near-vertical sea cliffs at the coast, the scarps are gently sloping, somewhat irregular features. They merge and bifurcate and occasionally rise in a series of steps; they generally follow the regional trend of the land surface which rises toward the south. Scarps faces range in elevation up to 20 m and are soil covered.

(ii) A low-level, near-horizontal scarp, termed the '5m scarp', is found around the northern coastline of Tongatapu and borders Fanga'uta Lagoon (FU) (Figure. 2.5 (b)). It is in the order of 3 to 8 m high and typically occurs at the contact of older limestone with unconsolidated Holocene sediments at its base. Its upper surface usually has a well-developed soil cover but on its face soils are thin or absent.

Source: Information modified from Roy (1990).

2.5 Tongatapu limestone

The Tongatapu block is made up of Pliocene-Pleistocene reef limestones presumably during Pliocene or Pleistocene times according to Lister (1891), Davis (1928) and Cunningham-Anscombe (1985). Underlying this are Pliocene and older volcaniclastic layers (Cunningham and Anscombe, 1985). Packham (1985) suggested the uplift and tilting occurred during the Miocene period coupled with a tilting rotation of about 1m/km based on his observations of Tonga's water well data.

Table 2.2 illustrates the time scale involved Subdivision of the Neogene Period when these formations occurred.

Table 2.2. Subdivision of the Neogene Period illustrating where the timescale co-relation between the Pleistocene, Pliocene and Miocene Epochs. (Source: wikipedia.org/wiki/Pliocene, 2015)

System/Period	**Series/Epoch**	**Stage/Age**	**Age (Ma)**
Quaternary	Pleistocene	Gelasian	younger
Neogene	Pliocene	Piacenzian	2.588 – 3.600
		Zanclean	3.600 – 5.333
	Miocene	Messinian	5.333 – 7.246
		Tortonian	7.246 – 11.608
		Serravallian	11.608 – 13.82
		Langhian	13.82 – 15.97
		Burdigalian	15.97 – 20.44
		Aquitanian	20.44 – 23.03
Paleogene	Oligocene	Chattian	older

The exposed limestones discussed above extend along Tongatapu's windward coastline, formed in a crescent shaped ridge, running from the southwest (SW), around the most southern (S) point of the island, to the southeast (SE) (refer Figure. 2.5 (b), 2.6 (a) (b) (c) & (d)).

The ridge formation around SE, S and SW sides corresponds to the former reef and the central low area is part of the original lagoon bed as per Roy (1990). It is the oldest palaeoreef on Tongatapu (Perrin and Mansergh, 1989). The intermittently distributed hills are from ancient patch reefs (Figure. 2.6) that grew during the time Tongatapu was just an atoll as described by Roy (1990).

Some of these limestone hills and the abandoned quarry site 'Ahononou, identified in the quarry section, have been mined for Tonga's infrastructure projects. "The only significant patch reefs that have not been mined are close to the Royal Palace on the Nuku'alofa waterfront and on the Royal estates at Kauvai" (Lewis, Smith and Pow, 1991).

Harrison (1993) and Tappin (1993) concluded their assessment of Tonga's limestone aggregate that "most of the aggregate are weak, dirty, porous and low durability". The soft nature of the limestones and the lack of adequate processing systems results in the generation of large quantitates of fines (quarry dust) and dirty aggregate products. The harder limestones are often not processed efficiently, the lack of processing resulting in large accumulations of quarry waste."

Limestones buried underneath the thick soil in the lagoonal areas are expected to be of generally the less inferior type limestones. That is, they are porous soft friable and rubbly algal bound limestones by Harrison (1993).

2.6 Quarry sites

Introduction

Based on the above observations, conclusions can be drawn that other than minor intrusions in the island of 'Eua next to Tongatapu, there are no local volcanic rock deposits in Tongatapu. Limestone is abundant throughout but the quality varies dependent on where they are found. Cotter et al (2010) established that the best quality, densest and hardest limestones are found in elevated areas to the south of the island and these qualities deteriorate gradually towards the north. This coincides with the work done by Roy (1990) and Harrison (1993).

Figure. 2.5 & 2.6 illustrates the elevated areas around the southern side of Tongatapu (Points A, B & C) and it is not a coincidence that the best quality aggregate can be sourced from the quarry on the most southern tip of the island, Point B in Figure. 2.6.

2.6.1 Quarry locations and land ownership

There are 12 quarry sites in Tongatapu (Table 2.3), 7 of which are operational whilst the other 5 are either non-operational due to exhausted limestone aggregate, excavation breached the water table or legal mining boundaries had been reached.

Location of quarry sites are not strictly based on the availability of good aggregate. The land tenure system influences the location process as well as the volume available. Mining rights negotiations can be complicated, lengthy and costly which in turn hinders the process.

As McCotter et al, (2010) stated that land ownership practices in Tonga are extremely complex and can complicate land use options. Land, as defined by the Land Act 1903 of Tonga, includes all land and its resources such as biodiversity and minerals (Pelesikoti, 2003). The overall ownership of the land 'technically' belongs to the King. This includes the hereditary estate of the king, the estates of the rest of the royal family, the hereditary estates of the nobles and titular chiefs (*matapule*) and government land. Land availability is a challenge for Tongatapu in particular due to the increased internal migration from outer islands (discussed in Chapter 4).

The considerable increase in the number of males that turned sixteen years old since 1996 fulfilling only the agricultural (bush allotment) provisions of the Land Act would have required 50% more land than exists in Tonga – 90, 214 hectares of land, versus 59,130 hectares of land judged suitable for dwelling or agriculture (MAF, 1999). It is estimated that 75% of eligible males are without a tax allotment (MLSNR, 2000). This is an issue central to the goal of sustainability in Tonga and in small island countries (Pelesikoti, 2003).

Those that inherit the land also control all resources in that allotment including minerals, if there is any (Pelesikoti, 2003). Pelesikoti also sighted that increasing number of people without land contributes towards non-management of resource usage which leads to non-formal lease arrangements creating sustainable land use management problems.

The agricultural or bush allotment provision and the control of resources contained within are the key issues when considering quarry locations. In order to access minerals or resources such as limestone aggregate, negotiating with multiple landowners is often the most difficult barrier to overcome. Hovering above the 'landowner' are layers of government, titular chiefs, nobles, members of the royal family and the king, to be consulted at some point in order to fully endorse any form of agreement.

2.6.2 Quarries

Accessing quarry sites is an issue for various reasons, particularly difficult terrain around the southern cliffs high above sea level, between points A and B shown in Figure. 2.6 . 'Ahononou Quarry, which contained the best quality aggregate, is located on the edge of the southern cliffs. This was a dormant site due to mining boundary being reached and the water table exposed.

The seven operating quarries are scattered throughout and closest to the south Pelehake/South Malapo operation located much further inland from ‘Ahononou.

McCotter et al (2010) independently tested aggregate from the above-mentioned quarries in 2010 as part on an Institutional Assessment of Road Construction and Maintenance Services study in the Kingdom of Tonga. In Chapter 6, these results are discussed together with test results carried out as part of this research.

Indicative locations of the above quarries are marked in the Figure. 2.8. Active quarries are marked in blue and inactive quarries in green.

In the next chapter, pavement design requirements and current road construction practice in Tonga are reviewed against limestone aggregate quality available to deliver projects.

Table 2.3. The 12 quarries in Tongatapu where some are currently in operation and some are inactive (modified from McCotter et al, 2010).

	Quarry Name	Owner/Operator	Associated village
1	Royco Quarry	Royco Amalgamated Company Ltd	Kahoua
2	Halaloto Quarry	Vete Holdings Corporation Ltd	
3	Tafolo Quarry	Tafolo Group of Companies	Lomaiviti
4	Farm Quarry	**Not operational**	Makeke
5	Pili Quarry	Nishi Trading Company Ltd	Pili
6	MOW Quarry	Ministry of Works **Not operational**	'Ahononou
7	Nishi Quarry	Nishi Trading Company Ltd	Longoteme
8	Malapo Quarry	Luna'Eva Enterprises	Malapo
9	Pelehake Quarry	Opened for Chinese funded projects	Pelehake/South Malapo
10	Nukeleka Quarry	**Not operational**	Nukuleka
11	Navutoka Quarry	**Not operational**	Navutoka
12	Kolonga Quarry	Lord Nuku **Not operational**	Kolonga

Figure. 2.8. Quarry locations in Tongatapu (Modified map from MOW, 2012)

Chapter 3

EMPIRICAL PAVEMENT DESIGN
CURRENT PRACTICE IN TONGA

This chapter focuses on the principle of pavement design as well as construction practices currently used in Tonga. The requirements for unbound flexible pavement (FP) types used in Tonga, will form the basis of background for discussions and in particular, aggregate strength characteristics that are suitable for road construction.

Unbound FP background is provided using the Australian (AUSTROAD) and New Zealand Transport Authority (NZTA) design standards discussing structural layers used for this method and how the relative strength of insitu subgrade material dictate design.

NZTA and AUSTROAD standards are being used to assess aggregate selection because Tonga does not have road construction standards. Road construction criterions used for Tonga projects are dictated by the donor or project funding countries. New Zealand and Australian standards have been used in the past hence it was deemed feasible to use these as reference points.

Relevant existing literature on empirical design is discussed highlighting specific aggregate design criteria. There are references to the mechanistic aspect of empirical design commenting on how traffic loading and resultant stresses, particularly relating to stresses, strains and modulus, impacts on pavements layers and how this is dispensed.

Behaviour of the aggregate under load and how presence of water in pavements affects its long term structural integrity are also discussed.

3.1 Pavement design

3.1.1 Background basics

Pavement is defined as the portion of the road, excluding shoulders, placed above the subgrade level for the support of, and to form a running surface, for vehicular traffic (AUSTROADS – Pavement Design Manual, 2015).

Design engineers are required to establish the route (geometric design), specify materials and determine pavement layer thickness in order to deliver its intended purpose and functions. To accomplish all this, numerous factors are to be considered. These include the following, modified from AUSTROADS (2015):

- Topography of the land
- Existing soils and their respective geotechnical properties
- Weather patterns including rainfall and temperature variations
- Traffic load and various types of vehicles
- Primary use
- Design life - whether the requirements is for high or low access
- Availability of construction materials
- Appropriate technical skills for construction

In addition, adopting an overall pavement life cycle strategy is essential where initial capital investment cost and maintenance costs are factored in. The entire life cycle of new pavements needs to be considered.

The initial overall cost benefit analysis of projects isn't part of this work but it is important to point out that constructing new pavements affects government agencies, road users, the environment and businesses.

Analysing the life cycle cost of various pavement options such as unbound granular pavement, stabilised pavement, structural asphalt or rigid pavements including cost impact during the construction phase would take into account the availability of sound aggregate material. Hence availability of appropriate pavement construction material is a critical part of the life cycle cost analysis.

Cost of the selected pavement option should also take into account the implication on road users during construction. Pidwerbesky (2015) presented that economic impact such as increase in vehicle operation costs, travel time delays which affects travel efficiency, petrol consumption, and prolonged environmental impact exposure needs to be evaluated.

When assessing life cycle costs, other risks are to be considered such as pavement structure failure and consequences, social and environmental impacts during maintenance, economic impact of ongoing maintenance, and risk of pavement surfacing failure (Pidwerbesky (2015).

Pavement design can be complex and variable depending on a number of factors. However, a simple comparison between industrialized and developing countries like Tonga shows that, depending of economic capacity, one is more complex than the other, illustrated in Figure 3.5. The simpler design based on the traditional empirical approach, as used in Tonga, is still the most cost effective method for developing countries.

This simplified approach forms the background, the technical level and method of pavement design discussed in this project. References will be made to some of the relevant mechanistic principles behind the empirical method identifying specific criteria in relation to aggregate material requirements focusing on AUSTROADS Sections 6 and 8.

Illustrated Figure 3.1 is a pavement design system flowchart relationship between the input variables, the analytical aspects as well as the decision making processes that constitute pavement design from AUSTROAD (2010).

AUSTROADS (2010) section 5 is relevant to this work as it determines the background information requirements for sections 6 and 8. It is however not included in the analysis part of this work but referenced throughout design discussions.

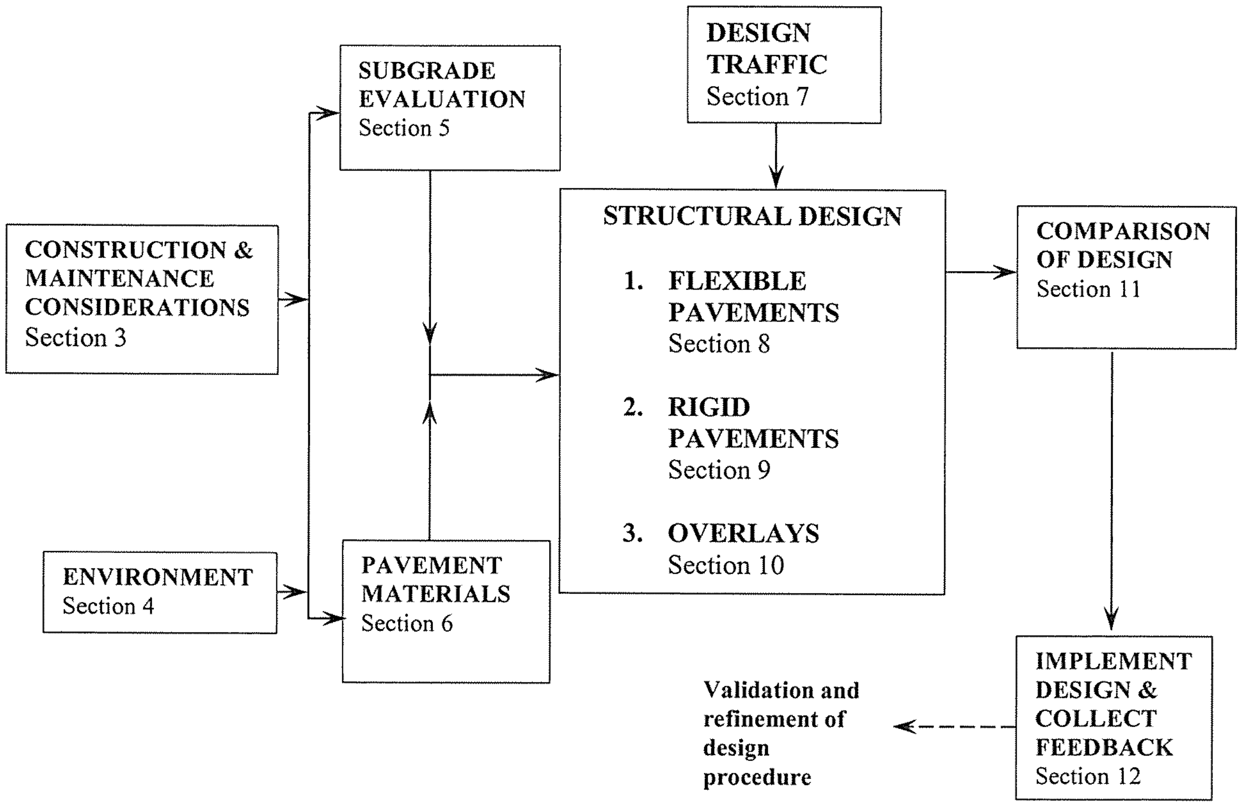


Figure. 3.1. Pavement design system for new pavements showing the relationship between pavement design factors. In practice however, some are combined or omitted altogether (AUSTROADS, 2010).

3.2 Typical road pavement cross sections

The types of pavement structures observed during research site visits in Tonga are a mixture of normal and light duty (refer 3.2.). The entire roading infrastructure is made up of a two-lane network.

Typical cross sections referred here are from NZTA standards (TNZ, 2002) providing background of a typical road pavement similar to Tonga roads.

Pavement or carriageway of an urban road cross section below TNZ (2002) in the context of this research, is the area between kerb face to kerb face (Figure. 3.2). Pavement discussions are based on the material required to construct pavement layers within this area.

3.2.1 New Zealand standard

One of the reasons New Zealand pavement construction and testing standards was deemed appropriate as reference for this work is because NZ standards had previously been specified for road reconstruction work carried out in Tonga during 1994 and 1995. These projects are reviewed in Chapter 4 as case studies.

Figure. 3.2 is a typical two-lane urban road that can be found in and around Tonga's capital's CBD. There is generally an allowance for footpath throughout the capital's CBD but no footpaths when outside the capital. Figure. 3.3 cross section (TNZ, 2002) is typically found outside Tonga's capital.

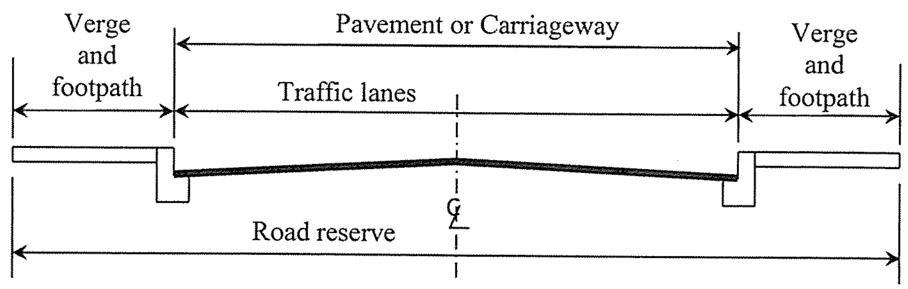


Figure. 3.2. A typical two-lane urban road cross section. The carriageway includes the associated drainage facilities such as kerb and channel. The road boundary is normally from property legal boundaries on each side of the road. (Source: Adopted from TNZ Geometric Design Manual, 2002)

The configurations of Tonga's highway and trunk road are very similar to the rural road typical cross section in Figure. 3.3.

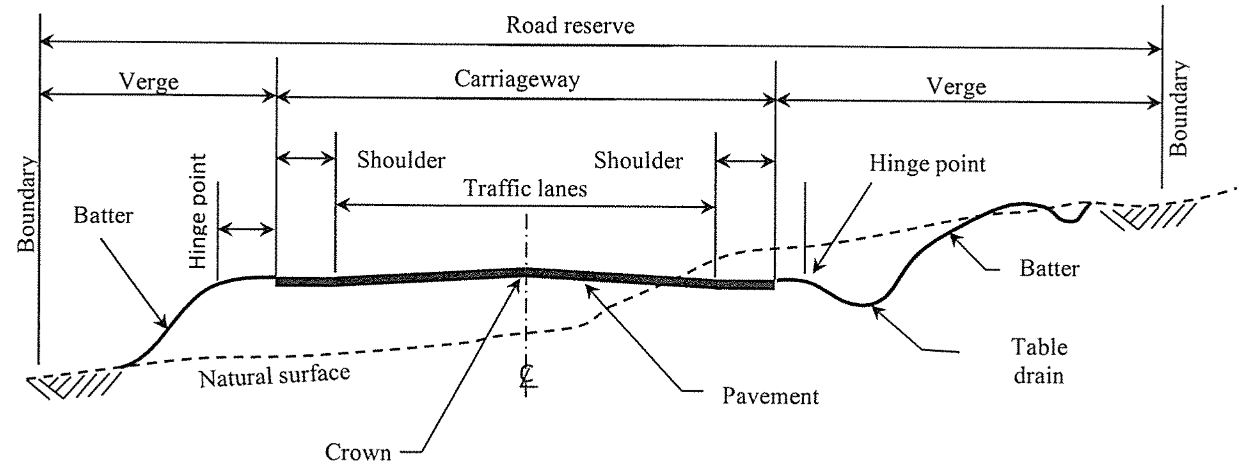


Figure. 3.3. Typical rural road cross section (Source: Adopted from TNZ Geometric Design Manual, 2002).

The main identifiable difference between the rural and urban road designs is the surface drainage system. There are no kerb and channels in the rural setup. Road boundaries are much more variable and wider. Shoulders are introduced for emergency stoppage and for foot traffic. Berms are not included but a verge exists between edge of carriageway and property boundaries. (TNZ Geometric Design Manual, 2002)

3.3 Pavement types

Background

Pavement structures are generally classified into two broad categories, Flexible and Rigid. Rigid pavements are constructed using Portland cement whereas flexible pavements are bituminous based surfacing pavements, covered in this work. Their applications are then divided into three types, based on current and future traffic volume, hence design requirements vary accordingly. The general pavement classifications are:

- **Heavy duty** – airport runways, specialised freeways (Autobahn), commercial ports, container terminals and forestry roads.
- **Normal duty** – motorways, local arterial roads, residential subdivision roads and commercial car parks.
- **Light duty** – temporary access roads, haul roads, recreational pathways, footpaths and playgrounds.

All of Tonga's roads consists of normal and light duty flexible pavements. "Dirt roads" form part of the road network but these are privately owned farm access roads only. Concrete pavements are utilised at ports and in parts of airports infrastructure.

3.4 Flexible pavements (FP)

Flexible pavement structures are considered a multi-layered system and their effectiveness can be impacted by many factors and importantly, the quality and properties of aggregate used. Poor quality aggregate can severely compromise design objectives, particularly if stress and strain effected by vehicular loading that results in high percentage of aggregate failure effecting irreversible plastic deformation.

Contrary to the rigid pavements, flexible pavement is considered an elastic structure that flexes to accommodate traffic loads when stresses are transmitted via vehicle tyres in contact with the pavement surface (Figure. 3.6).

FP surfaces are predominantly bituminous that can either be chipsealed or asphalt concrete surfacing. These may or may not incorporate underlying layers of stabilized or unstabilized granular materials on a prepared subgrade. These pavements can be made up of the following combinations.

- Thin bituminous seal over unbound granular
- Thin bituminous seal over unbound granular in cemented
- Thin bituminous seal over cemented material
- Thin surfacing over bitumen treated bituminous base over unbound granular
- Full depth asphalt
- Asphalt on unbound granular
- Asphalt on unbound granular on cemented material
- Asphalt on cemented material
- Asphalt on modified material

Source: Pidwerbesky (2015)

Historically pavement design was based on empirical information by observation of existing pavements and experimenting through full scale trials. In most developing countries, this is the most cost effective approach. Wealthier and more advanced countries have over the last decade or so have gradually been moving towards a more mechanistic approach whereby analysing the pavement as a structure and estimating material properties required to deliver the design specific for that area.

Chart solutions utilised for empirical design are still available that have been developed from historical mechanistic analyses.

This is the current method and adopted approach in Tonga hence the design focus will be on the AUSTROADS and NZTA empirical method, identifying the strength and weaknesses of the aggregate used.

3.5 Flexible unbound granular pavement (FUGP)

The makeup of pavement layers may vary from country to country and complexity reflects funding levels and/or the country's economic position. Araya (2011) demonstrated a basic comparison between developing and industrialized country pavement layers illustrated in Figure. 3.5.

Industrialized countries have much thicker surfacing layer, no need for base layer, and a capping layer that is sometimes stabilised using lime or cement is used above subgrade. Developing countries on the other hand have thinner surfacing, therefore the need for a basecourse layer, and no capping of subgrade.

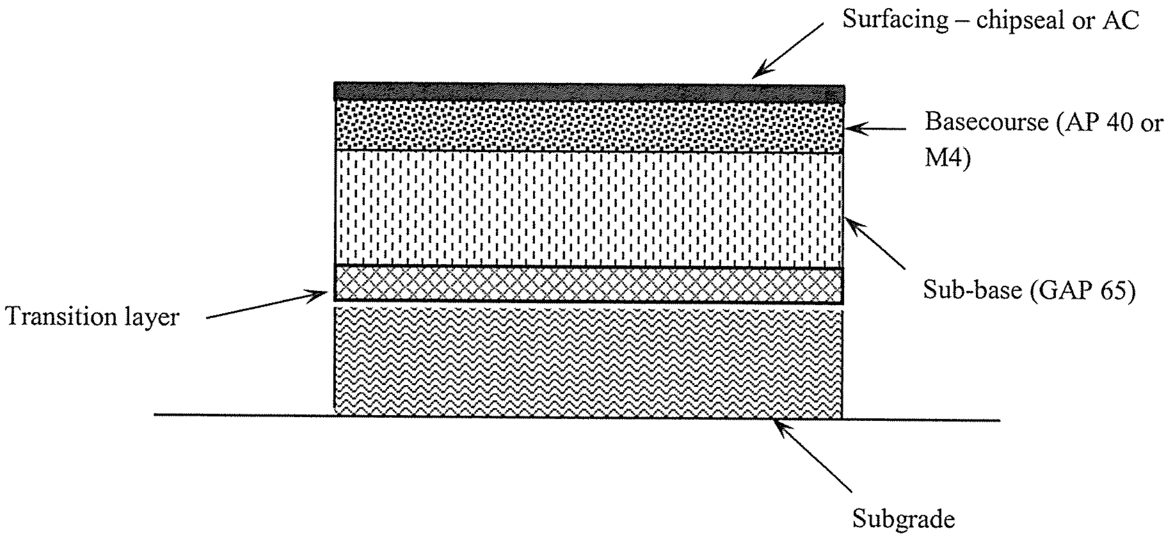


Figure. 3.4. Typical flexible pavement layout.

Unfortunately based on road re-construction observed and discussions with Ministry of Infrastructure engineers in Tonga during this research, fundamental design processes appeared to have been disregarded. Some of the possible reasons attributed to this are discussed in Chapter 7.

In Figure. 3.6 illustrates the typical chipseal depth observed in Tonga obtained during site visits, when existing surfacing were scarified to be renewed as part of Tonga road improvement project 2014. Depths varied between 5mm – 10mm. Excessive amount of fine dusty particles can be observed between crushed aggregate.

AC resurfacing work did not occur at the time of research site visits hence no physical AC samples were available. However, numerous AC potholes located throughout Tongatapu shown in Figure. 3.7. Typically, a depth of 50mm were specified in previous AC surfacing projects in Tonga which will form part of the discussions in Chapter 4.

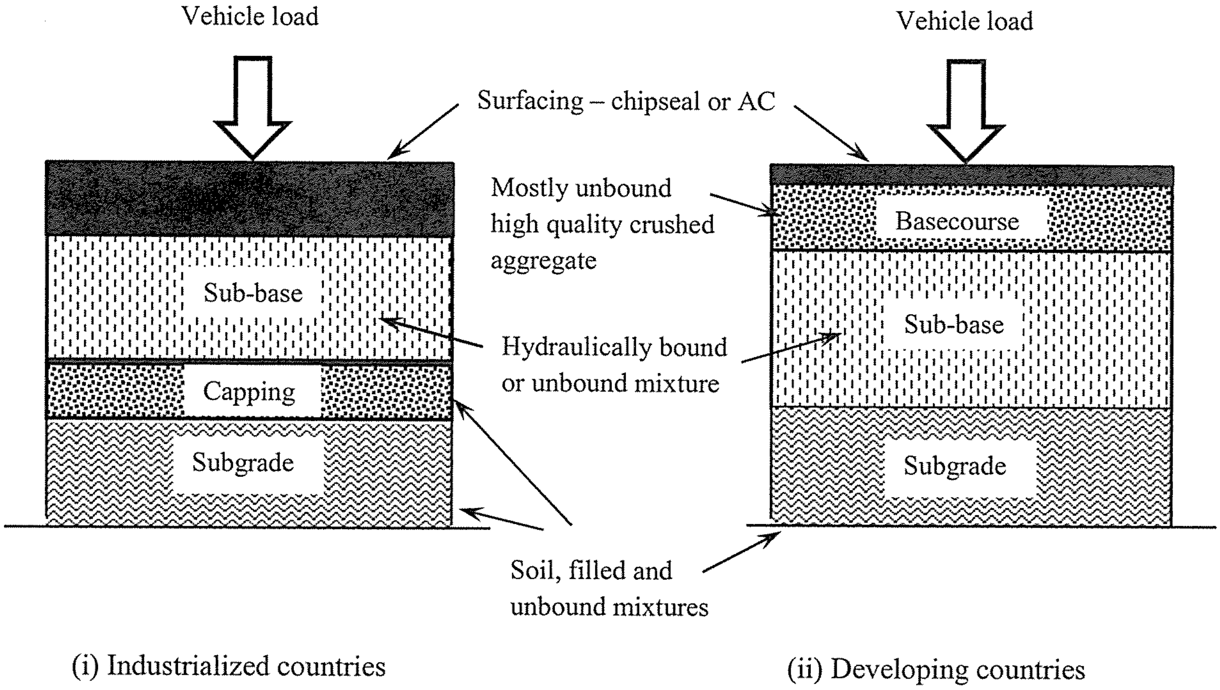


Figure. 3.5. Illustrating the typical pavement cross-sections from industrialized and developing countries (Modified from Araya (2011)).

Developing countries (ii) typically have thick sub-base and base (basecourse) and a thin surfacing layer for economic reasons. In countries like Tonga, chipseal surfacing is used for the majority of its network.

Figure. 3.6. Typical chipseal surfacing depth sample excavated during the 2013 reconstruction near Nuku'alofa CBD (Source: Hiliau, 2014)

Figure. 3.7. Typical asphaltic concrete potholes in and around Nuku'alofa CBD (Source: Hiliau, 2014)

3.6 Function of FUGP layers

The main purpose of the FP layers is to economically reduce the stresses imposed on the pavement structure to an acceptable level which can be supported by the sub-grade. Road base should be resistant to permanent deformation and cracking (or fatigue). The structure also support loads and generally, the pavement materials are stiffer than the in-situ material, dispersing the pressure from top to bottom, with the aim of reducing excessive subgrade ground deformation (Figure. 3.8).

Regardless of design approach, material availability or financial constraints, the principle of structural integrity, durability, experience and workability still applies. Hence the 'function' of any pavement determines the nature and main objectives of the design whilst maintaining essential elements of the pavement in providing:

- Access – adequate drainage under all climatic conditions. The combination of geometric design and pavement material can efficiently drain excess surface water away from the pavement reducing the chances of adverse consequences.
- Durable – structure to be sufficient to carry traffic loads
- Safe – this includes sufficient surface friction. Also required are adequate skidding resistance for vehicle braking purposes, low noise generation and favourable visibility through the use of lighting at night time.
- Surfacing riding quality by providing a smooth surface of an acceptable specified quality. This surface would be much smoother than the existing in-situ material and it contribute towards minimising vehicle operating costs.

An important design requirement is the acquired knowledge of existing and estimated future traffic loading, change in climatic conditions, and insitu geotechnical properties of exiting soils must be conducted as part of the design process. This does not appear to be a high priority in Tonga probably due to economic reasons, lack of local expertise and limited time for project delivery.

Table 3.1 summarises pavement layers and their various functions followed with some more detailed description.

Table 3.1. Typical functions of various flexible pavement components.

FUNCTION	Wearing Course	Basecourse	Sub-base	Filter Layer	Subgrade
LOADING	Load bearing and distribution	Load Distributing	Load Distributing	Low	Load Bearing
STRENGTH	Highest	High	Lower than basecourse	Low	Lower than sub-base
PERMEABILITY	Impermeable	Medium	High	High	Requires drainage protection
DURABILITY	Hard wearing. Resistant to oxidation & skidding	Top Quality granular aggregate TNZ M/4	Second Quality Granular Aggregate	Filter aggregate or Geotextile Fabric	Improved from natural state by drying and re-compaction or stabilising
PLASTICITY	Non Plastic	Non Plastic	May have some plasticity		May be plastic

3.6.1 FUGP general description

Typically flexible pavement layers from the top down, comprised of:

- Surface wearing course - Seal (10 -30mm), Asphalt layer (thickness can be 10mm up to structural depths of 400mm)
- Road-base /Base course (100 - 175mm)
- Sub-base (150 - 400mm)

- Sub-grade Improvement Layer - Capping Layer (200 - 300mm)
- Sub-grade/Formation (Can be in-situ material, re-compacted or imported fill)

3.6.2 Surface Wearing course (Surface Dressing)

Provides a smooth impermeable skid resistant surface. Chipseal is a common low cost surfacing option in NZ and Australia and various asphalt surface options such as Dense Graded (DG), Stone Mastic Asphalts (SMA) and the low noise Open Graded Porous Asphalt (OPGA) used on high volume roads, are available. Wearing course is not a structural layer although it is in fact under high stress.

3.6.3 Road-base (Basecourse)

This is the main structural layer of the pavement. Thickness depends on CBR (California bearing ratio) of sub-grade and traffic volume over life of pavement (ESA). Can be constructed from:-

(i) Basecourse - High quality aggregate - typical in New Zealand,

(ii) Bitumen bound Basecourse - BTBC- not uncommon in NZ,

(iii) Cement or lime bound (Lean-mix),

(iv) Continuous Concrete slab (Rigid pavement),

(v) Composite (i.e. continuously reinforced concrete with bituminous overlay), or

(vi) Cement or lime stabilised.

3.6.4 Sub-grade Improvement Layer

If strength of sub-grade is low (CBR < 5%) and susceptible to water (swells and softens) a selected material may be imported to improve the foundation. Alternatively the existing sub-grade soil is improved by stabilisation using Lime (CaO, $Ca(OH)_2$), Cement or KOBM (Kontinuous Oxygen Blast Maxiite) binder).

3.6.5 Sub-base

This is the lowest structural layer of the pavement which provides the working platform for constructing the road. The thickness of the sub-base depends on the strength or CBR of the existing sub-grade and traffic volume over the life of the pavement (measured in Equivalent Standard Axles or ESAs).

Usually sub-base material is made of dense unbound crushed aggregate, but can be open grading to improve drainage. The usual accepted checks are grading, making sure that the aggregate grading does not exceed 10% fines. The durability or soundness test ($MgSO_4$) also needs to comply.

3.6.6 Sub-grade

In-situ soil or fill compacted at or near optimum moisture content. This is the most significant part of the pavement. All pavement layers are designed to reduce the deformation or strain in the sub-grade. The CBR value is determined as part of the pre-project design geotechnical investigation work. A value which is < 4 is considered a low value and a CBR > 10 considered high value. The lower the CBR value (weak subgrade), the thicker the overall payment layer which equates to higher cost.

A subgrade improvement layer (SIL) is needed when the CBR value is low and this can be carried out by modifying the subgrade using lime or cement, Geogrids and/or Geofabrics materials or replacing the soil with rockfill (Pidwerbesky, 2015).

3.6.7 Strain Alleviating Membrane Interlayers (SAMI's)

Highly elastic SAMI layers (i.e. not brittle) which is highly resistant to cracking, is used to resist reflective cracking when re-sealing. SAMI products are available in the New Zealand market and one in particular uses a combination of fibre glass and a polymer modified emulsion that produces a highly crack resistant membrane.

Between the surfacing and the in-situ or fill compacted subgrade soil (defined as the top 1.0m in TNZ F/1:1997) are subbase layers split into the sub-base and basecourse layers.

An additional layer can also be included in the structure known as a transition layer (Figure. 3.4), particularly if water tables are relatively high or the area is susceptible to high seasonal water levels. Also described as filtration or drainage layer.

3.7 Pavement loads

The resultant pavement stresses through the application of a wheel load are illustrated in Figure 3.9. Maximum vertical stresses occur directly under the wheel point of contact. At the point of contact between wheel and surfacing, maximum vertical compressive stress occurs. Pavement layers are designed to distribute this compressive stress decreasing from the surface (P_0) down to the subgrade layer (P_1) refer Figure. 3.8.

A complex interaction of stresses applied via wheel loads to the pavement bounded layers as the vehicle travels along the road. As the vehicle moves, the strain causing compressive stress is distributed down to the subgrade, illustrated in Figure 3.8, continually from under the wheel contact point. At the same time, tensile strain is generated at the top of these bounded layers just behind and in front of the wheel as it rolls forward. Tensile strain is also occurring simultaneously at the bottom boundary of the bounded layer(s), at the top of unbounded aggregate later, causing fatigue.

In Figure. 3.9 Garber and Hoel (2009) have illustrated a transition point where the effect of compressive stresses reduces to a point where the forces acting on the material changes from compressive strain to tensile mode, referred to as the neutral axis.

Vehicle accelration and braking effecting shear on the pavement structure which produces shear stresses adding to the above stresses actng on the pavement structure.

Therefore to recapitulate, compressive stress acting vertically on the pavement is dissipated throughout the pacvement structure down to the subgrade. Tensile stresses are cerated through pavement deflection and is found only in the bound layers. Shear stresses are produced shear in the pavement and can cut through surface bounded to the granular basecourse material.

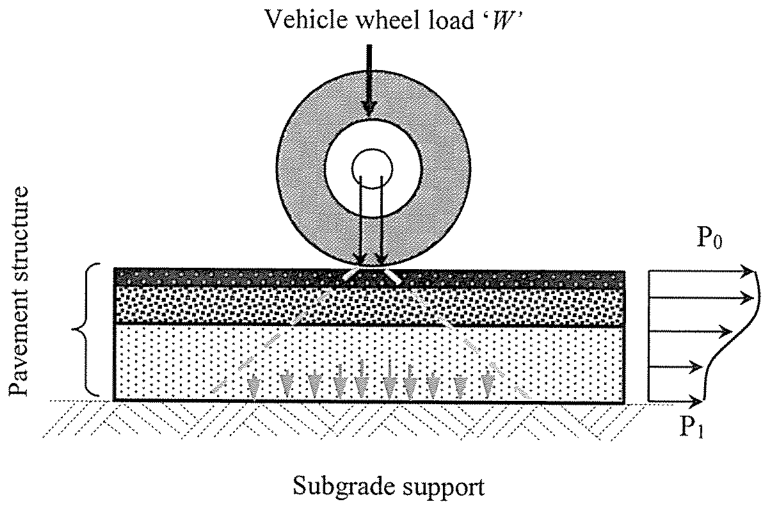


Figure. 3.8. Wheel load ***'W'*** is applied resulting in compressive stresses on the pavement structure (Adopted from Transit NZ, 2005).

Compressive stress P_0 is distributed through the sub-base layers is reduced to a level that causes minimal subgrade deformation represented by P_1 above (Transit NZ, 2005).

Accumulation effects of these forces over time can result in irreversible subgrade and pavement damage, such as wheel track rutting or shallow pavement shear failure. In some cases it can result in complete pavement failure. The likelihood of failure occuring depends on three types of critical strains generated by wheel load stresses. These are elastic, recoverable and residual strains.

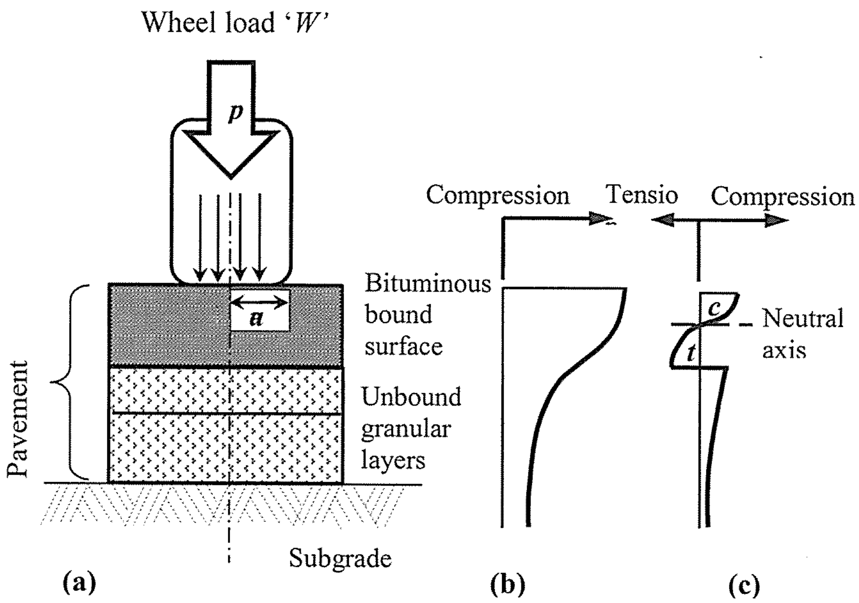
If the pavement material rebounds back 100% to it's original position after being subjected to vehicle loading stresses, this is referred to as elastic strain. Recoverable or resilient strain refers to the portion of strain that rebounds after a load is removed and is not 100%. The thrid type is residual or permanent strain which is the main cause of eventual pavement falure due to accumulation.

A combination of compressive and tensile stresses in asphalt bound surface material (Figure 3.9). Compressive stresses occur at the top surface under the wheel load, with tensile stresses occurring below the boundary between basecourse and surfacing layers.

Stress calculations are more precise when applied to more predictable engineering materials such as steel beams where scope of geometry alignment and homogeneity of material composition are relatively consistent and defined. The resultant forces and behaviour of components can be more predictable compared to flexible pavement.

Applying mechanical theories and calculations to pavement structures is not as straightforward as the structural steel scenario because a number of factors might or might not be consistent for the entire scope of design work. Subgrade material are tested then treated to ensure consistency in material behaviour when combined with the pavement aggregate material above it. That is the challenge presented for mechanistic pavement designers to resolve.

Good quality aggregate materials for pavement construction is crucial in delivering mechanical theory design expectations and establishing effective maintenance long term.



(a) Pavement layers
(b) Distribution of vertical stress under centreline of wheel load.
(c) Distribution of horizontal stress under centreline of wheel load.

Where:

p= wheel pressure applied on the pavement surface
a = radius of circular area over which wheel load is spread
c = compressive horizontal stress
t = tensile horizontal stress

Figure. 3.9 Distribution of stresses within Flexible Pavements (modified from Garber and Hoel, 2009)

Werkmeister (2003) demonstrated how the grain structure of unbound granular material (UGM) are affected by the various stresses where repeated load triaxial tests results shows when aggregate pavement material is under stress, deformation corresponds to the area under the hysteresis loops, Figure. 3.11. The work being done during the deformation process is converted to heat which leads to an altered state of materials involved, hence the material's eventual breakdown.

Other researchers have worked on the permanent strain repeated loading triaxial tests (RLT) attempting to predict the magnitude of permanent strain based on specified loads and stresses. What has been illustrated here is only a simplistic view of the entire process but enough to have an insight into the causes of material deformation as this is quite a distant outside the scope of this research.

There are many other key relationships in order to fully describe the permanent strain behaviour which can be referred in Arnold (2003).

It has been demonstrated during RLT tests by Arnold (2004) that there's a small amount of permanent strain incurred in every load cycle although it is not very clear when looking at Figure. 3.11. The accumulation of permanent strain causes eventual pavement failure.

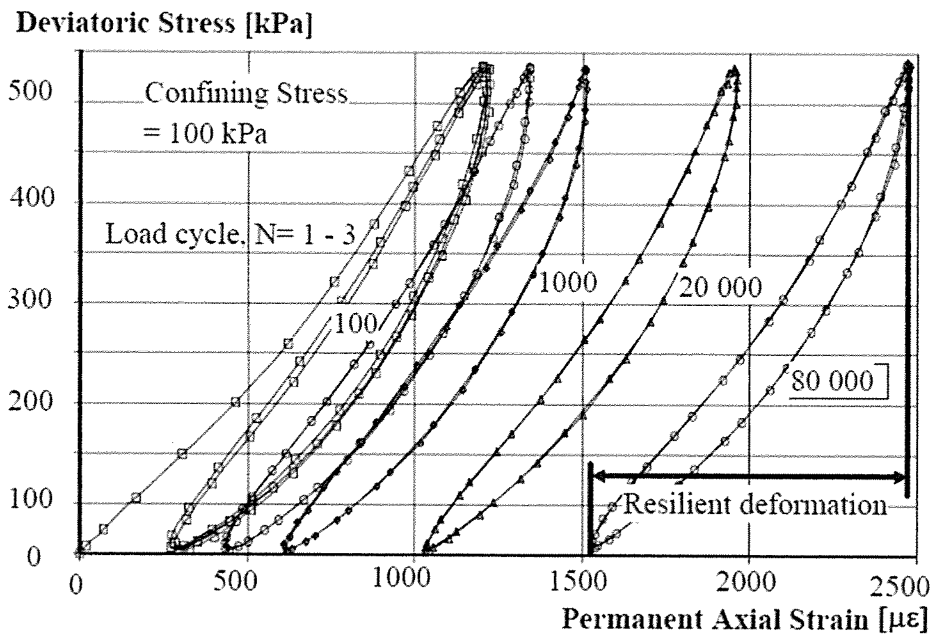


Figure. 3.10. Stress-strain behaviour of materials under repeated loading (Werkmeister, 2003) and (Arnold, 2004).

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This leads onto discussing the empirical method which is experience focused where outcome is predicted based on learned results from work carried out on similar ground conditions whilst taking all environmental factors into account.

3.8 Design Methods

Historically pavement design was based on empirical information obtained through observation of existing pavements and experiments using full scale trials. In developing countries, this is the most cost effective approach (reference?). In recent times wealthier and more advanced countries have gradually moved towards a more mechanistic approach whereby analysing pavement as structures then specify material properties required to deliver based on these specific designs criteria. In other words, more detailed analysis of existing conditions is becoming more common than relying on experience and historical data.

The chart solutions utilised for empirical design developed from historical mechanistic analyses are still being used and in many countries still effective (reference?).

This is the current method and adopted approach in Tonga hence the design focus will be on the AUSTROADS and NZTA empirical method in identifying the strength and weaknesses of the aggregate to be used.

3.8.1 Empirical

Empirical approach is based on extensive field observations of pavement performance under various conditions then an empirical correlation is established between traffic loading, material property and pavement layer thickness including subgrade material strength, and environmental conditions.

An example of this is the method developed by the American Association of State Highway and Transportation Officials (ASSHTO, 1993). Field tests conducted by ASSHTO up to 1942 (Jameson, 1996) and pavement design curves developed as a result. These were subsequently adopted by Australia's and New Zealand's respective roading authorities, AUSTROAD and NZTA, adjusted for local conditions which became the common pavement design guide (refer to Figure. 3.10).

The main weakness of this approach is its reliance on the limited site specific conditions empirical relations. Empirical solutions can be invalidated by changing parameters such as axle loads, tyre pressure or the use of different material for other purposes such as modified surfacing material or maintenance material overlays during the pavement's designed lifetime.

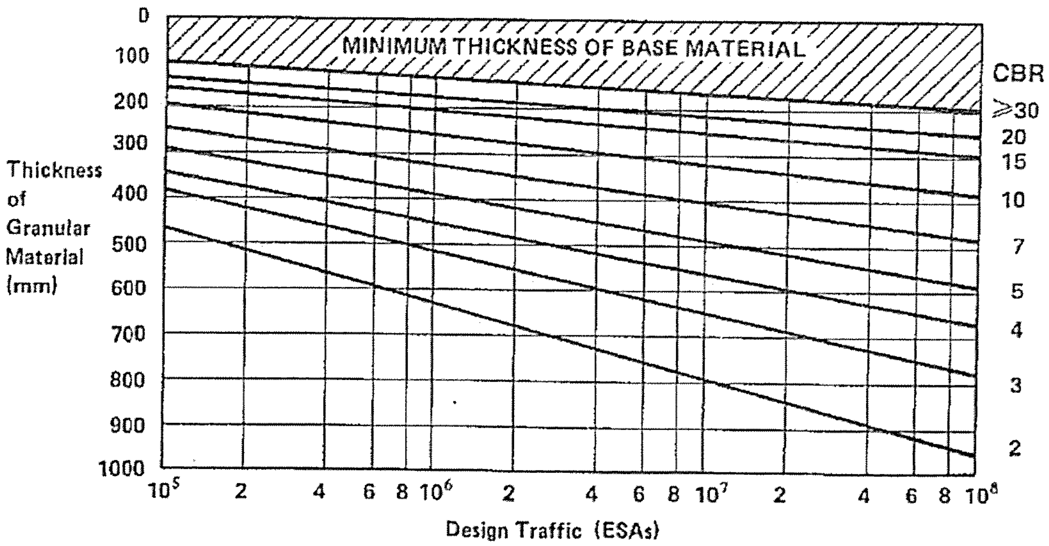


Figure. 3.11. AUSTROADS design chart for granular pavements with thin bituminous surfacings (AUSTROADS, 2004).

3.8.2 CBR thickness design method

As mentioned above, the strength of existing subgrade material dictates the depth and aggregate requirements to be placed on top. Combining subgrade strength, measured as a CBR value, with predicted wheel loads, the thickness of aggregate to be placed on top of the subgrade are determined from charts.

The original CBR oriented method did not consider load applications and quality of the overlaying material. The focus of this method was ensuring that compressive stress applied on top of the subgrade did not result in elastic strain being exceeded therefore avoiding accumulation of permanent strain which normally leads to deformation. Further information on this method can be sighted in Araya (2011).

Araya (2011) stated that the AUSTROADS Guide Figure 8.4 chart, Figure. 3.11 which is extensively used for empirical pavement design, originated from the CBR pavement design method. The AUSTROADS guide evolved to rely mainly on the critical responses based on the maximum horizontal tensile strain at the bottom of the surfacing layer, asphalt or cement bounded, and the maximum vertical compressive strain at the top of the subgrade, Figure. 3.8.

3.9 Aggregate behaviour

Many arguments have been presented to highlight the most important aggregate properties that would affect unbound pavement performance. A comprehensive study of aggregate test methods by Saeed et al., (2001) of unbound aggregate performance highlighted a number of factors that contribute toward unbound pavement poor performance. These were aggregate shear strength, gradation, density, fines content, moisture level, particle angularity and surface texture, material degradation during construction and repeated loads, drainability and freeze thaw cycling.

Aggregate properties determined by Saeed et al., (2001) to be responsible for unbound aggregate sub-base layer performance were shear strength, frost susceptibility, durability, stiffness and toughness.

In this research however, it was only possible to review shear strength or crushing resistance, gradation or particle size distribution and weathering. Test for samples suitability as chipsealing aggregate was also carried out.

3.9.1 Unbound granular material performance

The following discussions will be focused on relevance to limestone aggregate utilised in Tonga which is the empirical method with mechanistic design included for any relevant clarification particularly AUSTROADS and NZTA/TNZ aggregate requirements

For a given subgrade CBR (or modulus) combined with a design traffic loading (equivalent single axels (ESA)) a minimum thickness of unbound aggregate layer is prescribed. Two of the primary mechanisms of fatigue failure or distress to design against are:

- **Subgrade (permanent) deformation**
  Vertical compressive strain generated in the subgrade resulting in rutting; and

- **Strength and durability of the aggregate**
  Unbound aggregate materials are required to resist loading, except for the drainage layer, by having inherently strong properties so to absorb and distribute the stress induced loads to acceptable levels. Durable material will not be adversely affected during construction and in fact attain high compacted modulus which in turn will be effective for long term subgrade protection.

The effect of vehicle loading is distributed throughout pavement layers via tyre pressure changes is an important aspect using a simplified static tyre load highlighted by Huang (2003) and Yoder & Witczak (1975).

After considering factors such as tyre pressure, contact pressure, tyre wall pressure etc. two assumptions were made. The pressure on the road surface equals to tyre pressure, therefore they equal to the vehicle load. The load is assumed to be tyre contact area multiplied by tyre pressure. The contact area is assumed to be circular with a radius.

When the load is applied with static surface contact pressure and the pavement layer depth increases, the vertical compressive stresses decreases. If however the load is increased and the tyre pressure remain constant, vertical compressive stresses on the road surface remains the same whilst stresses increases in deeper layers.

Therefore load affects deeper pavement layers while tyre pressure affects specifically the surfacing. Thickness of layers is a function of vehicle loading and the surfacing is a function of tyre pressure.

This does raise the importance of all layers being able to carry out their functions effectively to cope with variable pressure and loading exposed to. It also raises the importance of having robust material for chipsealing especially in Tonga due to the extensive use this type of surfacing throughout.

Further discussions will take place further relating to this in Chapter 6.

3.9.2 Unbound Granular Material (UGM) deformation

Stresses that UGMs are subjected to, thousands and in some cases millions of load cycles, results in either the pavement is in a permanently strain state and some bounce back to their original state, resilient strain. Therefore Figure 3.10 illustrates the stress-strain relation which is non-linear and upon removal of the load, it returns forming a hysteresis loop. This is only a single loop but analysis of hysteresis loops would produce the permanent and resilient strain for each cycle.

The strength and interlocking ability of pavement aggregate plays a key role in absorbing the stress whilst maintaining its structural integrity. Araya (2011) stated the work done by Thom and Brown (1988) to demonstrate that crushed limestone grading with high fines content reduces the aggregate's stiffness and resistance to permanent deformation. Araya (2002) also determined that aggregate grading within the specified limits has a major influence in resistance to permanent deformation.

Discussed above is when vehicle travels along the pavement surface, vehicle load is applied vertically as compressive stress, tensile stress due to deflection, and shear stresses when braking or accelerating in variable magnitudes.

The wheel load and stresses are transferred by grain-to-grain contact of the aggregate through the pavement structure, illustrated in Figure. 3.12.

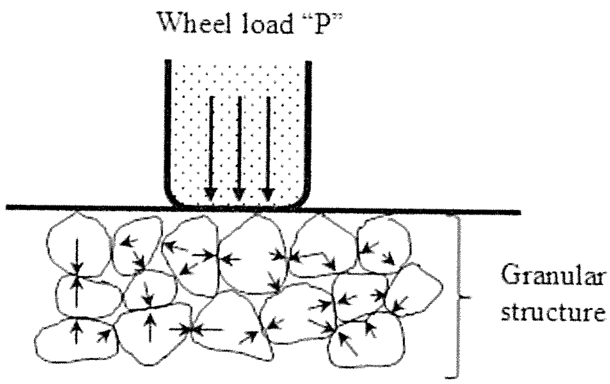


Figure. 3.12. An illustration of load transfer in granular pavement material (Hiliau, 2018)

Stress dependant nature of UGM also has to include the dead weight of the material, Molenaar (2009). Historical stress and dead weight of material acting on pavement elements, minus the traffic load, are referred to as confining stress.

Arnold (2004) discussed how major and minor principle stresses increase simultaneously when load approaches and element or aggregate particle. A rotation of principle occurs whereby the stresses rotate around the centre of the element. Taking into account that aggregate particles are presumably static and not able to roll when load is applied, from a shear stress perspective, vertical and horizontal stresses increase and shear stress increases until the wheel load is directly above the element or particle.

When the load is directly above the element, according to Arnold (2004), vertical and horizontal shear stress levels equals to zero and when the load moves away, reversal of shear stress occurs. This leads on to what occurs within aggregate grain structure in reaction to the stresses when load is applied so that a correlation can be drawn between this and what appears to be a key factor associated with Tonga's limestone aggregate problems.

Resilient deformation is caused by individual aggregate grains deforming as per Werkmeister (2003). Due to the fact that particles are in direct contact within the pavement structure, when loads or forces are increased, particles are compressed together and inter-particle connections transmit this force from one particle to the next resulting in an increased inter-particle contact area, and therefor increasing the resistance from adjacent particles, as shown in Figure. 3.12.

In densely compacted material, similar to the Tongan situation with high level of fines, shear strain forces particles on top of each other causing an increase in volume. There is no room for expansion so the dilation will therefore result in increased stiffness of the pavement layer.

Werkmeister (2003) demonstrated how particle resilient deformation or Displacement ($\Delta\delta$) decreases when contact force ΔF increases and at point "0" no stress exists when F equals to 0. As the contact forces increase, pressure on individual particles also increases, pushing aggregate particles into available spaces within the mix. This can result in high crushing resistance aggregate re-orientating or disintegration in low crushing resistance aggregate.

Lekarp (1997) described the mechanism of re-orientation as the main cause of permanent deformation. Initially particles rotate and slide around to re-orientate under applied pressure. Particle grain breakages occur then the stresses between grains exceeds the inherent strength of the material which is influenced by grain size, stress magnitude and mineralogy.

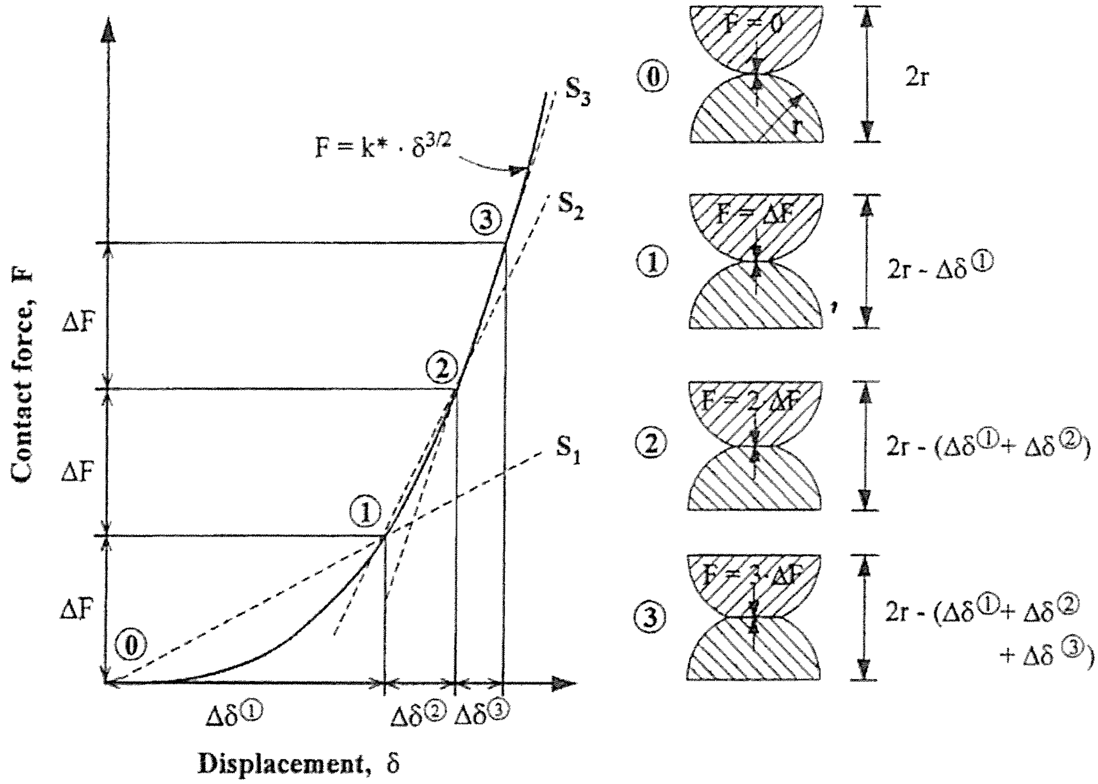


Figure. 3.13. Dependence between the contact force F and displacement σ between two particles (Source: Werkmeister, 2003).

Thom and Brown (1989) found that shear strength permanent deformation resistance is a function of particle roughness. Through assessing elastic stiffness, shear strength and proneness to deformation by the use of RTL tests, they discovered that elastic stiffness correlates with frictional resistance at particle contact points, which depends on microscopic properties.

Visible particle roughness is the function that causes shear strength to resist deformation. There were some connections between shear strength and resistance to permanent deformation however it was argued that shear strength is influenced by overall particle shape and roughness.

More research work is needed to determine the exact connection between shear strength and permanent deformation due to variable stress levels detected between individual particles when subject to static or dynamic loading.

3.10 Water in pavements

One of the main problems confronting pavement engineers worldwide is water intrusion into UGL. Effective drainage systems are an integral part of a well performed highway or road infrastructures. Not only it benefits pavement structural integrity but it is also provides a safe driving surface for motorists.

Some water in the UGL can be beneficial to the strength and the stress-strain behaviour of pavement layers and is required for optimum water level and achieving optimum shear strength. Figure. 3.19 from Werkmeister (2003) shows RLT test results of two samples starting with the same moisture content and one of the sample is drained whilst the other sample remained saturated. The undrained sample increased permanent deformation to a much higher level than the drained sample when subjected to same number of stress cycles. Suffice to point out that excessive amount of water saturating pavement structures would have detrimental effects on the long term integrity of such pavements.

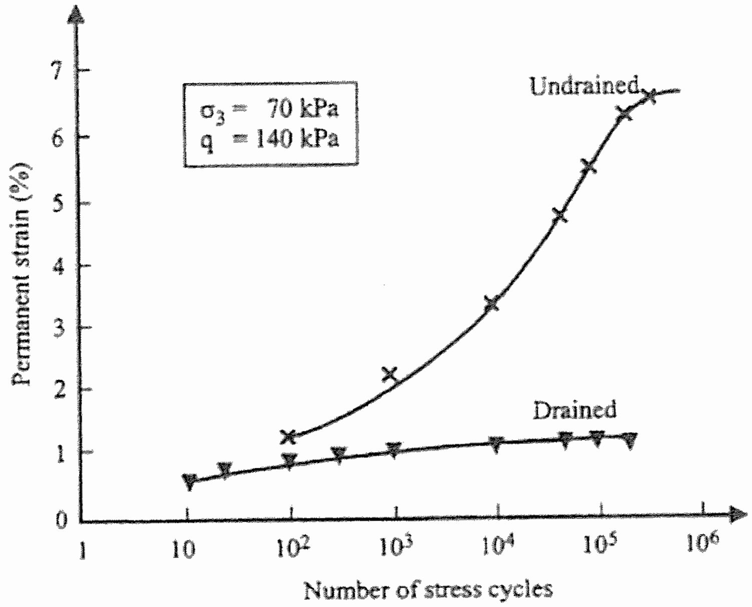


Figure. 3.14. Drainage influence on permanent deformation (Werkmesiter, 2003)

Tonga and the Pacific islands are subjected to months of high intensity rainfall events during the historically rainy season from January to April. During this period, low lying districts including the capital Nuku'alofa, suffer from flooding and pavements remain saturated during this period.

Drainage infrastructure is very limited and private properties are treated as the road drainage exit points. In other cases stormwater runoff is drained into the harbour or estuaries leaving these areas polluted, unswimmable, and marine life extinguished in many places.

Tonga's existing road infrastructure is presented in Chapter 4. Three of Tonga's historical road construction projects are also reviewed where international technical assistance requested by the Government is the normal avenue for funding, designing and constructing projects. Specification of aggregate for pavement construction will be the main focus and any relevant design details will also be presented.

Chapter 4

TONGAN ROAD INFRASTRUCTURE

There are two parts to this chapter. Part 1 introduces population, population growth dynamics, density and subsequent transport demand on Tonga road infrastructure. Also included in this part is a review of Tongatapu's road infrastructure highlighting flexible components which relates to design principles discussed in Chapter 3.

The second part to this chapter is a case study of three roading construction projects from 1994/1995, 2009, and 2010/211. This will highlight the aggregate test requirements selected for this research.

Specific requirements for aggregate quality assurance, focusing on aggregate testing aggregate extracted from the relevant sections of the contract documents for discussion.

4.1 Public Transportation Systems

Public Transportation Systems is a generic term for services available to societies by linking various urban and rural locations. It is a combination of traditional and new technology mobile services that exists in modern civilisations encompassing regions, a nation or the whole world.

There are three types of transit modes described as Mass transit, Paratransit, and Rideshare (Noel, 2009).

Designing roads to provide adequate service for these transit modes are influenced by a number of factors such as vehicle design, speed, road user characteristics, functional classification, traffic volume by type, e.g. % of Heavy Commercial Vehicles (HCV), and bus network role and bus passenger numbers.

In Tonga's situation, it is a simple network made up of fixed routes (Mass transit) for buses and limited heavy commercial vehicles combined with a very flexible personalized service (Paratransit) of car rentals, private taxi owner-operators and limited other specialized users.

4.1.1 New Zealand Road Network

New Zealand's state highway (SH) classification is based on "functional requirements" such as moving freight to and from ports or between main centres (Mass transit). This is further broken down to categories described by NZTA (2011) as national strategic, regional strategic, regional connector, and regional distributor. The threshold levels for all the different categories are based on type and number of vehicles users and these have been established as category level 1, 2 and 3.

Prioritising the above classification based on their significant contribution toward economic growth directly assist investment decision making at government level. NZTA (2011) affirms that funding SHs is based on investment and revenue strategy where activities are prioritised based on the following three key points.

- Strategic fit - The extent to which they address key opportunities from a national perspective
- Effectiveness - How well they achieve particular outcomes identified in 'strategic fit', and
- Economic efficiency - How efficiently the resources are being used

Therefore there is a direct connection at policy level between highway classification-level of service-road design and the requirements to deliver an expected level of service (LOS). It is an important connection between highway classification and levels of service provided for users such safety, travel time, and reliability which then feeds into the design decision process which is highlighted by NZTA (2011).

Complex cities like Auckland classification of road types are broken down further to Strategic routes – connecting towns and cities, Regional arterials – connect primary, District arterials – connect major parts of the city, Collector roads – connect parts of the city, and Local roads – connect adjacent properties.

These are followed by specific geometric design requirements based on traffic volumes (Annual Average Daily Traffic or AADT), traffic composition, topography of land and speed.

It is recognised that different road user categories share and compete for the same carriageway space. So in more complex cities, transport plans with higher level objectives to improve traffic flow typically may include: Getting the best out of the system, reduce car dependency in cities and better provision for freight movement (Espada & Green, 2015).

Increasing level of traffic congestion is a challenge for cities worldwide particularly due to an escalation in migratory movements driven by economic disparities resulting in people seeking employment where opportunities exist. New Zealand is one of the countries accepting migrants to a level that infrastructure is under unprecedented pressure to meet the demand.

To cope with the increasing demand on the road infrastructure, new roads are built, realigned or widening existing of the existing has traditionally been the approach to managing this problem both in New Zealand and Australia. Similarly, Tonga's mainland is being subjected to similar pressures. There is limited land space and an extreme lack of financial capability in Tonga, therefore it doesn't have the capacity to continually supply new roads when required.

4.1.2 Managing traffic

Making the most and getting the best out of existing infrastructure could be utilized as the main traffic management approach. Traditional measures used resulted in minimal increase in LOS for current or future traffic levels without substantial infrastructure investment. It is essential to have enough space to provide for HOV (high occupancy vehicles) and public transport priority measures.

Supply side strategies to achieve the strategic directions highlighted by Espada and Green (2015) typically include a number of relevant points but those that are more practical and applicable to this research are hierarchy of use in the network, Kerbside parking management and optimisation of peak flows on major arterials.

4.1.3 Vehicle use

The main factor to be considered in all road design is the type of vehicles that are likely to use the proposed pavement therefore geometrical and structural pavement design are to reflect intended use. The four general vehicle classes specified in New Zealand's NZS 4404 similar to the types used in Tonga are:

- Semi-trailer (17.0m) or B-train (20.0m)
- Single unit truck or bus (12.5m)
- Medium single unit truck (8.0m)
- Car (5.0m).

"A road pavement must be wide enough and of suitable geometry to permit all vehicles to safely operate at an acceptable speed. In addition, it must be strong enough to cater for both the heaviest of these vehicles and the cumulative effects of passages of all vehicles" (AUSTROADS, 2008).

Mixed traffic use all roads is common throughout unless it is physically impossible for vehicles to fit into the available carriageway space or they are regulated against it. The key is recognizing and making allowances for this in practice as illustrated by Brindle (1987) in Figure 4.1.

Arterials for example require high mobility function but very low in access function to avoid disrupting mobility. In contrast, local streets require high access function and network mobility function is low as high speed in these areas affects foot traffic safety.

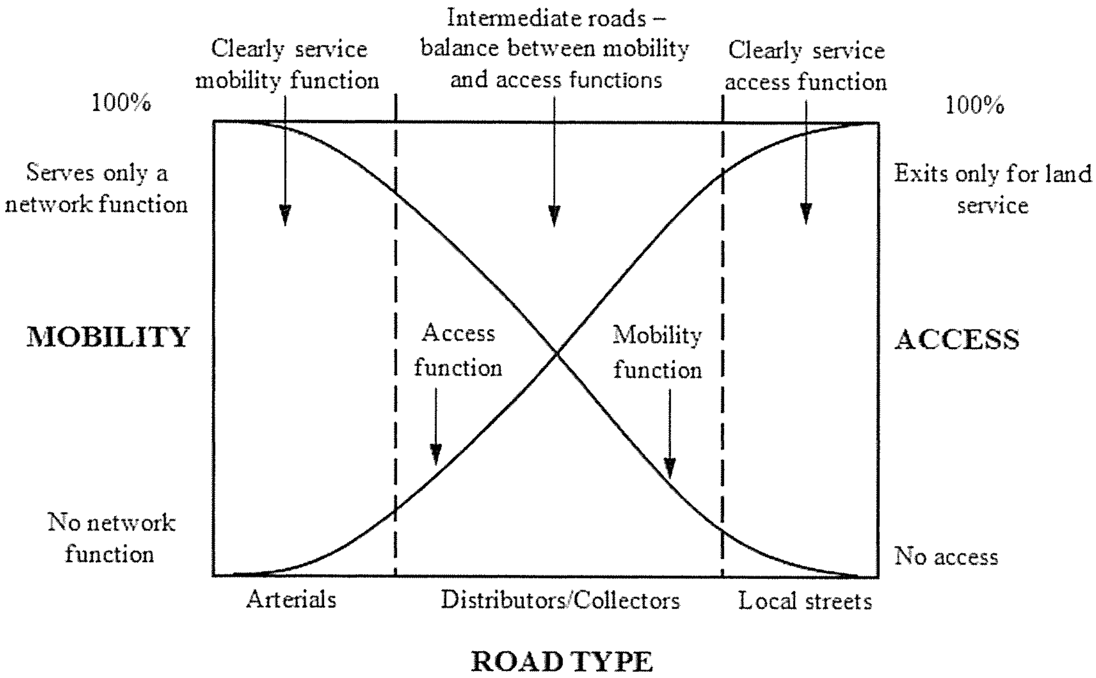


Figure. 4.1. The reality of road function and different types (Source: Brindle, 1987).

The above functionality is supported by AUSTROADS Guide to Pavement Technology definition of road functional class as: "The main purpose of defining a road's functional class is to provide a basis for establishing the policies that will guide the management of the road, by grouping roads together into categories according to their intended services or qualities" (AUSTROADS, 2015).

4.2 Tonga's Population

The most recent census conducted for the Kingdom of Tonga was in 2011 and the first was taken in 1921, though there is population data from 1891 of < 20 000 from Pelesikoti (2003). Census took place every ten years from 1956 onwards with exception of the 2006 and 2011 being only five years apart. The 2011 census was carried out in order to bring the census information up to date prior to Tonga's November 2014 General Election, requested by the Electoral Boundary Commission (EBC).

In 2011, Tonga's population was 103 252 compared to 101 991 in 2006, which is an increase of just over 1%. Seventy three percent (75 374 people) of the total population reside in Tongatapu, which is an increase from 2% over five years. Tongatapu's population density in 2011 was 290 ppl/km^2 compared to 159 for the entire country and 123 for the next most populated island of Vava'u (Table 4.1).

Table 4.1. Tonga's population density by island division for 1996, 2006 & 2011 (Source: modified from Lolohea & Koloamatangi, 2014)

Division	**Land area (km^2)**	**Population density (ppl/km^2)**		
		1996	**2006**	**2011**
TONGA	**650**	**150**	**157**	**159**
Tongatapu	260	257	277	290
Vava'u	121	130	128	123
Ha'apai	109	74	69	61
'Eua	87	56	60	57
Ongo Niua	72	28	23	18

Over the last 110 years, Tonga's population increased by 82 552, which equates to 80% of the population in 2011. There was an increase of 16% between 1901 and 1956 (55 years), a 39% over the next 20 years, then a 25% increase between 1966 and 2011 (45 years), refer Figure. 4.2 (Lolohea and Koloamatangi, 2014).

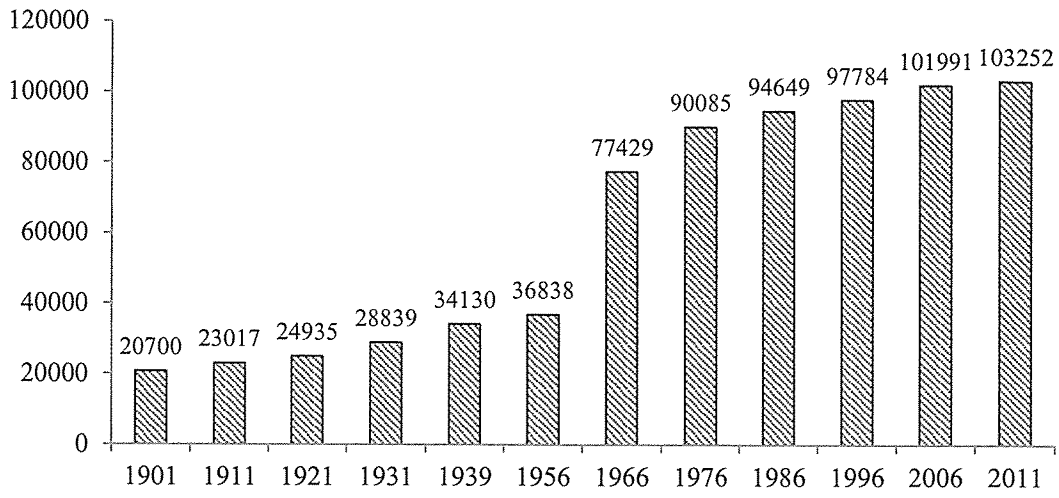


Figure. 4.2. Tonga's population between 1901 and 2011 (Source: Lolohea & Koloamatangi, 2014).

Focusing on infrastructure needs perspective for Tongatapu, the overall rate of population growth for Tonga between 2006 and 2011 was 0.2% whilst Tongatapu the mainland where most employment opportunities exist, increased by 0.9%. All other islands showed population losses during this period. In fact Lolohea and Koloamatangi (2014) report stated that population declined in the outer islands between 2006 and 2011 at a higher rate than the period 1996-2006 refer Figure.4.3.

The overview of the road infrastructure situation is the Tongatapu with an increasing population, based on the 2011 census, is currently experiencing a lot of pressure due to congested roads with limited capacity to accommodate such demand.

Internal migration is a significant contributor to increasing demand and stress being applied on Tonga roads particularly on the mainland, Tongatapu. A number of the aforementioned reports are reviewed, increasing vehicle numbers, and population growth are explored.

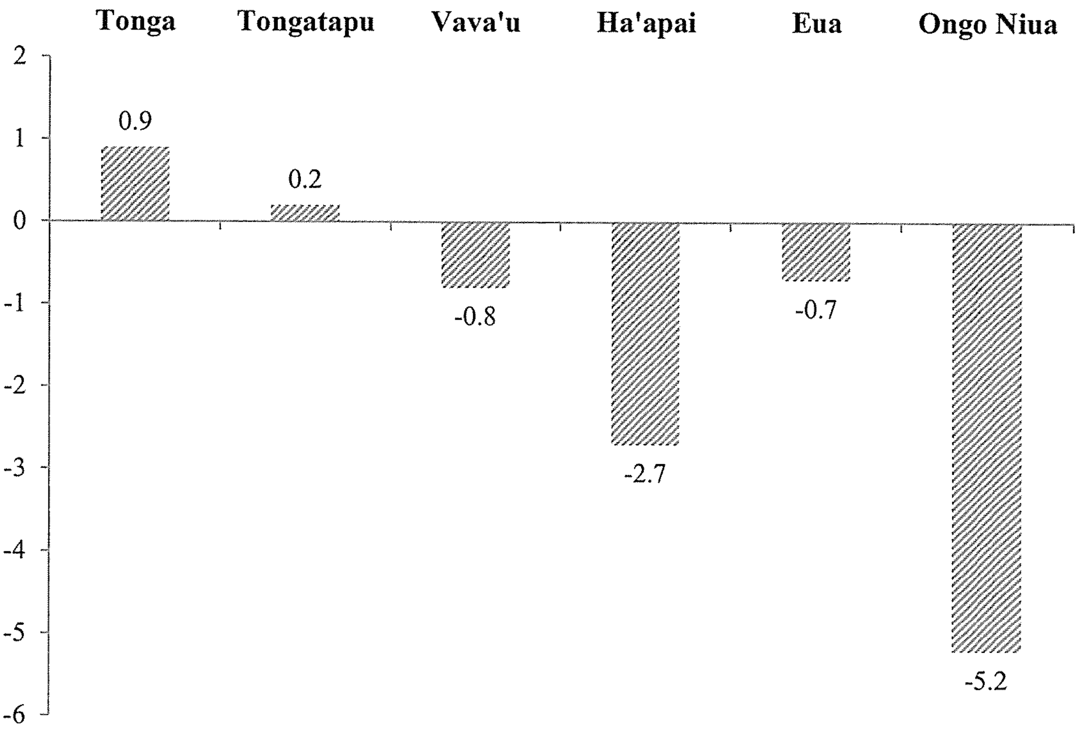


Figure. 4.3. Tonga's average annual population growth rate (%) by division for 2006 – 2011 (Source: Lolohea & Koloamatangi, 2014).

4.3 Demand on infrastructure

The statistical analysis in Figure. 4.3 emphasises the fact that internal migration affects Tongatapu significantly more than all other islands. Increasing population would certainly equate to an increase in vehicular numbers. Tongatapu have limited land space for road expansion would intensify the demand on an already frail, inadequately maintained road infrastructure.

Historical transport infrastructure and road safety reports are reviewed here to illustrate the trend in increasing demand and pressure on Tonga roads over the years therefore future mitigation planning could have commenced much earlier. These reports were the most recent work done on these specific issues which further reinforces the view that technical up to date knowledge is lacking in all public transport related areas.

4.3.1 Vehicle import trend

All vehicles in Tonga are imported and the numbers have been increasing for decades on a parallel trend to internal migration. There was a lack of recent comprehensive data on the vehicle importation numbers for Tonga so again historical information is used in this section to illustrate the trend in this sector. Reviewed data shows that on average, 83% of licensed vehicles are located on the mainland Tongatapu, refer Table 4.2, which supports the allusion that Tongatapu's road infrastructure is increasingly under pressure.

Table 4.2. A summary of vehicles licensed in Tonga from 1980 to 1989. Highlighted is the percentage of vehicles located in Tongatapu (Source: World Bank, 1993).

Island Divisions	**1980**	**1985**	**1986**	**1987**	**1988**	**1989**
Tongatapu	N/A	2800	3352	4033	5339	3644
'Eua	N/A	91	72	68	130	118
Ha'apai	N/A	84	85	83	100	98
Vava'u	N/A	395	446	554	629	797
Niuas	N/A	24	21	20	25	24
TOTAL	**2849**	**3394**	**3976**	**4758**	**6223**	**4681**
Tongatapu (%)	**N/A**	**82.5**	**84.3**	**84.8**	**85.8**	**77.9**

MOW (1994) (need to add the 1994 MOW contracts as references) reported as part of the road reconstruction 1994/95 scope of work that 82.5% of vehicles were in Tongatapu in 1985, increasing to 85.8% in 1988, which is consistent with the World Bank (1993) report.

It is clear and well accepted that vehicle numbers are increasing every year. It was estimated in 2014 by MOW engineers during research site visits that on average there were 300 new vehicles are registered on a monthly basis.

In this scenario, Tongatapu is receiving almost. 3000 new vehicles per year which consistent with reported numbers from World Bank (1993) and MOW (1994) (need to add the 1994 MOW contracts as references).

Table 4.3. Registered vehicles data between 1985 and 1990 (Source: MOW, 1994)

VEHICLES	**1985**	**1989**	**1990**
Cars & light trucks	1823	2307	3010
Heavy trucks	495	895	965
Taxis	277	691	1032
Buses	111	95	118
Motor cycles	392	473	501
TOTAL	**3098**	**4461**	**5627**

4.4 Tonga' road Infrastructure

An essential part of this work involved reviewing existing road infrastructure database to establish background knowledge of how Tonga maintain the quality and long term integrity of its roads. Infrastructure reports have been carried out over three decades on the island's road network by international consultants in partnership with relevant ministries.

Whilst attempting to gather information and data during this research, a central contact point or database didn't exist and had to endure a very vague unsystematic process in accessing reports. This is definitely reflected in the country's fragmented maintenance of its road infrastructure network. How the government approaches funding for road construction projects, pavement design, management systems, and construction standards also reflects the overall ad hoc approach at ministry level.

4.4.1 Road classifications

Tonga's road classification is based on a very simple hierarchy of network, refer Table 4.2. This is in direct contrast to systems in developed countries like New Zealand and the AUTROADS highway infrastructure systems where multi-levels hierarchy approach is required to manage a complex network of roads.

As referred earlier, Tonga have two main road classifications consists of what they've designated as main roads or highways and trunk roads. Trunk roads are further divided into two sub-categories based on surfacing treatment, sealed and unsealed surfaces. Unsealed surfaces are constructed using coral or insitu ground material referred to as dirt roads. Functional details for these roads are listed in Table 4.2.

There are two more functional categories in Tables 4.2 and 4.3 referred to as feeder and access roads. There is no real clear design distinction between feeder roads and trunk roads. In reality, feeder roads are part of the trunk road network and they are made up of sealed or unsealed surfaces. Access roads are a mixture of dirt roads and coral rock surfaces connecting private properties or in some cases they lead towards trunk roads.

All responsibilities for managing and maintaining the road infrastructure rests with a number of ministries and local or regional councils are non-existent. There is a relatively complex network and cross related functions amongst ministries but the day to day civil engineering operations and maintenance responsibility lies with Ministry of Works.

World Bank (1993) funded Pacific Transport Sector Study of Tonga's Transport Sector discussed various contributing factors towards underperformance in transport. MOW's technical capability and level of responsibility was highlighted where expertise lacked in critical areas.

World Bank (1993) also noted that forward construction and maintenance planning for all public roads were Ministry of Work's responsibility. Managing the road and quarry operations at the time were three road engineers plus eleven technical and financial support staff. Again it was emphasised that the MOW is responsible for all public road construction and maintenance without any local village contribution due to lack of technical skills.

This clearly illustrated the lack of resources and expertise throughout the islands. Financing projects are driven by international aid bundled together with the provision of expertise to design, manage, and build projects eliminating the opportunities to develop local knowledge.

These internationally funded projects are prime examples of Tonga's reliance on foreign assistance.

4.4.2 Road network

In this section the inconsistency in data reporting of Tonga's infrastructure is highlighted based on various reports extracted from four different sources. There are too many variable amongst the reports to rely conclusively on one lot of data.

The Government undertook transport sector review, reported by Paterson et al., (2005) with the intention to improve the structure and efficiency of the transport sector including ports and airports. It concluded that the system was totally fragmented where eleven different agencies involved in managing transport, sharing the responsibilities between them. The fragmented nature of responsibilities amongst different ministries is reflected by the inconsistencies of infrastructure statistics being reported.

Prior to the MOW (1994/1995) road reconstruction projects, Japan's JICA Basic Design Team in 1988 carried out a survey of Tonga's infrastructure for design purposes.. The survey work recorded all road types including unsealed feeder and access roads including a general standard description, Table 4.2.

Table 4.4. Tonga road infrastructure functional classification compiled by JICA Basic Design Team in 1988 as part of the work toward Tonga's 1994/1995 road reconstruction project (MOW, 1994/1995)

FUNCTIONAL CLASSIFICATION		ROAD LENGTH IN 1988		ENGINEERING STANDARD
		Tonga Islands	Tongatapu	
Highways	Major trunk roads with high traffic volumes, linking significant urban centres or to airports	81.5 kms	64.6 km	Class A Sealed road Reserve width: 14.6m Formation width: 11.0m Sealed width: 7.0m or over
Trunk Roads	Roads linking all villages/ towns to the Highways and/or other villages, or strategically providing a linkage to another Trunk Road	363.0 kms	188.5 km	Class B Sealed road Reserve width: 14.6m Formation width: 9.1m Sealed width: 7.0m or over
Feeder Roads	Roads linking villages/ towns to agricultural area; roads between Trunk Roads or roads to public beaches and major institutions	666.0 kms	248.0 km	Class C Coral road Reserve width: 11.0m Formation width: 7.9m
Access Roads	Roads to individual 'apis (small groups of 'apis) or roads between feeder roads	679.0 kms		Class D Coral road Reserve width: 7.3m Formation width: 4.9m
	TOTAL	1 789.5 km	988.1 km	

At closer inspection the two ministries predominantly responsible for Tonga's road system were the Ministry of Works and Disaster Relief Activities (MOW) and Ministry of Lands, Survey and Natural Resources (MLSNR). Majority of the road infrastructure throughout the three main islands is found on Tongatapu where 68% of the total road network is found. In order to create a clearer overview or an estimate of Tongatapu's infrastructure, data from various report are summarised below.

There are anomalies amongst all with the exception of 1985 and 1988 data. The 2000 and 2008 Feeder and Access road Figures are well below data from previous years. The only possible explanation is that recent focus have been placed on Highway and main Feeder road upgrades due to lack of maintenance over the years. Prioritising the main roads

Table 4.5. This is an accumulation of road network data extracted from historical reports carried out for Tonga. Sources: Paterson et al,. (2005), World Bank, (1993), Fawcett, (2000), and MOW Tongatapu, (2008)

Tongatapu	**Highway**	**Trunk**	**Feeder**	**Access**	**Total (kms)**
1985					939.10
1988	64.6	188.6	248.0	487.0	988.20
2000	216.0		220.0	82.0	518.00
2008	266.1	99.31	122.2		487.61

4.4.3 Maintenance

The lack of a suitable road infrastructure maintenance system was an important issue raised in the World Bank (1989) report. The general poor state of Tonga roads at the commencement of this research (2012/2013) corroborated the 1993 World Bank report regarding the non-existence of any effective maintenance programme being applied. Figure 4.1 is a typical example of neglected chipseal Highway Roads during 2013 site visit.

Maintenance is a key component in extending the intended life of infrastructures such as roading. World Bank (1993) report raised the importance of maintenance specifically for future donor project to incorporate funding toward maintenance needs. It also recommended the review of the infrastructures existing condition then prioritise cost to carry out the maintenance work.

Figure. 4.4. Chipseal Highway Roads around the coast of Tongatapu had been left to deteriorate for some time due to lack of maintenance (Source: Hiliau, 2014)

The World Bank (1993) report also referred to the ambiguous and fragmented allocation of institutional responsibilities between government agencies as one of the constraints to improved performance and limited engagement of the private sector to drive competitive business also hinders performance.

Recent re-structuring of the Government ministries and more involvement of private sector reflects the above assertions by the World Bank. **It isn't clear however** whether the restructure improved the situation or not.

In addition to issues of underperformance pointed out by World Bank (1993), underlying traditional and cultural values have stifling effects on western world developed systems and this needs further investigation outside of this research.

4.4.4 Road Safety

Historical data and reports are utilised again, due to absence of recent studies in Tonga, to highlight another trend this time in road safety. The general view is that more traffic on the road equates to higher probability of having traffic accidents. Vehicle data above clearly shows Tongatapu have the highest number of vehicles therefore, this is reflected in vehicle accident statistics.

Road safety was first cited in 1989 as "an emerging problem" with 282 recorded accidents compared to 94 in the previous year (World Bank, 1989) (add to reference). The report also highlighted that "no systematic or continuing" assessment of the trend in accidents was carried out. An ADB funded report by Silcock (1996) assigning injuries to specific categories making it easier to assess the degree of seriousness. Data presented was from 1995. Significantly, reported accidents increased by 30% from 1994 to 1995; 287 and 374 respectively refer Table 4.6.

Table 4.6. 1995 road traffic accidents in Tonga based on region and injury severity (Source: Silcock, 1996).

Location	Fatal	Serious injury	Slight injury	Damage only	TOTAL
Tongatapu	10	14	58	238	320
Vava'u	1		5	33	39
Ha'apai	2	1	6	5	14
'Eua	1				1
TOTAL	**14**	**155**	**69**	**276**	**374**

In 1994, 12 people died as a result of road accidents compared to 14 in 1995 and of those 14 people, 7 were pedestrians. 10 of the 14 (74.1%) fatalities in 1995 occurred in Tongatapu. It was also revealed that 75% of traffic accidents in 1995 occurred around Nuku'alofa, Tongatapu (Silcock, 1996).

Silcock (1996) drew a correlation between the highest number of vehicles registered of 85.2% in Tongatapu, and the highest number of road accidents, 85.6% also in Tongatapu, Table 4.7.

Table 4.7. 1995 vehicle registration location comparison with road traffic accidents in Tonga (Source: modified from Silcock, 1996)

Location	**Tongatapu**	**Vava'u**	**Ha'apai**	**'Eua**	**Ongo Niua**	**TOTAL**
Total vehicles	14396	2002	232	238	30	**16898**
% vehicles	85.2	11.8	1.4	1.4	0.2	100
% accidents	85.6	10.4	3.7	3.7	-	100

From an international perspective, two universal statistics are used to assess relative road safety. This is done by comparing vehicle numbers, population and rate of fatalities. In 1995, there were approximately 180 vehicles per 1000 people in Tonga based on an estimated population of 95 000. There were 14 fatalities, a rate of 8.3 deaths per 10 000 vehicles.

In addition, a fatality index Figure is derived using the proportion of death, 14 (14.3% of 98) out of 98 (injured) for Tonga, commonly referred for international comparison purposes.

Based on the above measures, Tonga was considered to be in the middle range internationally but was much higher than the industrialised country fatality indices of 2 per 10 000 vehicles and a fatality index of 2%. Worst countries have rates of 50 per 10 000 vehicles and higher than 25% fatality indices. Tonga's 8.3 per 10 000 vehicles was slightly better than Fiji's 9.8 (1993) and Western Samoa's 11.3 (1993). Fatality index of 14.3% was much worse than Fiji (6.3% in 1994) and Western Samoa (9.7% in 1993).

Clarifying further an important contributing factor to Tonga's 1995 high road accident fatality record is that pedestrian involvement contributed to 50% of the fatalities. Of the 7 pedestrian fatalities, 5 occurred in Nuku'alofa where there is high pedestrian activity take around town and CBD.

Seven government ministries are responsible for road safety, namely Ministry of Police, Ministry of Works (MOW), Tonga Power Board, Ministry of Health, Ministry of Education, Crown Law Department, and the Ministry of Lands, Survey and Natural Resources.

None of the agencies involved have managed to carry out evaluation of road accidents costs. It is generally estimated that cost of accidents is equivalent to 2% of GDP in developing countries. Tonga's GDP in 1996 was estimated to be between $US150-200 million, making the cost of accidents between $US2-3 million per year (Silcock, 1996).

The formal road safety auditing system proposed by Silcock (1996), incorporating local traditional cultural protocols, would lead to more efficient and improved infrastructure safety long term.

Education and effective enforcement programmes would assist in fundamental management of road safety discussed extensively by Silcock (1996). Silcock (1996) also argued that the lack of competent local traffic engineering personnel to manage traffic management contributes directly to existing problems.

Chapter 5

CASE STUDY OF MATERIAL SPECIFICATION OF 3 HISTORICAL TONGAN PROJECTS

Following on from the numerous internationally funded reports and reviews of Tonga's infrastructure, the Government, with further assistance, established framework objectives in order to action recommendations from these reports. NSPF 2009/2010 and the NIIP 2010 are essentially the next phase of the rebuilding framework process.

NIIP (2010) directed no additional priority projects over the next five years for roading other than to complete committed projects already underway. The main focus was to implement arrangements for sustainable road maintenance funding and delivery. The Government will "require advisory support of specialist technical assistance."

Prioritisation timing of projects depends on availability of donor funds. Specialisation and technical expertise are normally internationally supplied engaged through the donor countries or as per Government request.

The Government for example requested assistance from the Pacific Region Infrastructure Facility (PRIF) to ascertain a framework in an effort to address aggregate and road maintenance problems carried out as the Institutional Assessment of Road Construction and Maintenance Services in the Royal Kingdom of Tonga by McCotter et al., (2010).

The PRIF is a Sydney based partnership between the Asian Development Bank (ADB), Australian Aid (AusAID), New Zealand Aid (NZAID), European Union and the World Bank.

The Tonga National Road Improvement Project (2010) was funded via an assistance loan from the People's Republic of China to rehabilitate 119km in Tongatapu, Vava'u and Ha'apai. The contract specification was created according to Chinese road construction standards and some of the discussions regarding standard specifications will include references to this project.

In addition to the above main project, a portion of minor road works for 45kms was also allocated, funded as part of the AusAID/World Bank Transport Sector Consolidation Project (2010) with the intention that it will assist the local construction sector and local consulting industry to develop their skills.

The variability in design specifications continues with Chinese and Australian consultants working on the above Tongan projects. This leads on to highlighting and discussing of aggregate specifications from previously constructed roading projects.

5.1 Projects

Construction specifications from Tongan historical projects are reviewed in order to highlight any possible problems relating to unreliable pavement performance.

The intention was to simply identify then extract specific and relevant clauses from the project documents that had direct influence on determining aggregate material selection, aggregate tests and construction application.

The focus is in two parts. First is how the quality of aggregate at the source was determined and the specified test or quality assurance requirements ensuring aggregate was fit for purpose. The second focus is the contractual construction requirements of the unbound aggregate layers; sub-base, basecourse and surfacing.

Completed and archived contract documents were assessed and noted that some were marked "for tendering purposes" but these included relevant materials specification for the projects so these were suitable for this purpose.

Also to be noted that the road improvement projects 1994 and 1995 projects were designed, supervised and constructed by the same organisations so they were jointly assessed as one project in this research. The 1994 project continued into 1995 hence the different labels but essentially all specification were identical.

The third project listed was an endorsed contract between Tonga's MOW and China Civil Engineering Group Co Ltd. This project does not form part of the quality and performance detail discussion due to the design background being based on the "technical standards and specifications of highway engineering issued by the Ministry of Transportation of the Republic of China" (Contract No. ZTY01-09, 2009).

A statement in the Chinese produced contract documents, Contract No. ZTY01-09 (2009), referred to the "local conventional construction methods of existing highways" provided by the MOW was further evaluated. It was then established that "local road or highways design standards" did not exist and assumed that only verbal instructions from local engineer was possible. Therefore this error was taken into consideration and no references were made to local to local standards.

Subsequent research to obtain copies of the specified Chinese standards such as Technical Standards of Highway Engineering (JTG B01-203) and Specifications for Subgrade Design of Highway (JTG F10) proved to be beyond capability of this researcher. Copies of these standards were eventually located at MOW in Tonga but translation was impossible even the Tongan engineers did not understand any of it which also highlights some of the major quality issues faced.

However, for reasons unknown, some parts of the aggregate specification was written in English making it possible to present comments though limited, and further details can be obtained from the Chinese engineering standards.

Three projects:

1. The Project for Road Improvement in Tonga December 1994 and September 1995.
2. Tongan Integrated Sector Development – ADB funded project no. IUDSP 10-007, 2010
3. Tonga National Road Improvement Project – Contract No. ZTY01-09, 2009.

5.2 The project for road improvement in Tongatapu Island

Background to project

The Project for Road Improvement in Tongatapu Island was carried out in two phases, first phase commenced in December 1994 and the second in September 1995.

This work was to be carried out on Tongatapu, the mainland, where majority of the population resides. The road network was deemed "unsatisfactory, affecting adversely the transportation of agriculture products to the port, promotion of tourism industry and other socio-economic activities" (MOW, 1994).

Financial and technical assistance was requested by the Tongan Government from their Japanese counterparts. The Japanese Government responded positively by sending a study team from Japan International Coöperation Agency (JICA) in October 1993 to prioritise and quantify the requested work. Katahira & Engineers International of Tokyo of Japan were the consultants for both projects.

5.2.1 Project scope

The initial work Phase 1 consisted of upgrading Section 13.1kms of the main arterial routes to the capital's main port, Queen Salote Wharf. The scope of work involved widening, paving or overlaying existing surfaces with asphaltic concrete (AC), which was a new product to Tonga at the time. Some drainage improvement measures were also included to improve existing systems.

Phase 2 involved upgrading 21.0kms of the main road from the capital Nuku'alofa to Fua'amotu International Airport. The scope of work for phase 2 project involved upgrading the main road that carries all eastern and southbound traffic, connecting Nuku'alofa and the airport. It also caters for all international arrivals and departures.

Australian, New Zealand, Japanese or other equivalent road construction standards were specified as the compliance benchmark in the General Provisions for Codes and Standards. The powers to veto any material used rested with the consultant or the engineer engaged to carry out the design work.

5.2.2 Basecourse

Basecourse requirements for roadwork specified for this project was to be of coral pit derivative free of organic matter and clay. A grading envelope was presented, refer Table 5.1, and 100% of material to pass through the 50mm sieve, in other words a GAP50 aggregate.

Table 5.1. Basecourse grading envelope and all relevant test method. (Source: Project for Road Improvement in Tonga, 1994)

Item	Standard Sieve Size(mm)	Mass percent passing Grade (%)	Test Method
Particle size distribution % by mass passing AS sieve	50.00 37.50 19.00 4.75 0.425 0.0725	100 75-100 50-85 20-55 10-30 5-20	AS1289.C6.1 using wet sieving procedure
Liquid Limit	25 %		AS1289.C1.1
Plasticity Index	6 %		AS1289.C3.1
C.B.R	80 %		AS1289.E AS1289.F

5.2.3 Aggregate for asphaltic concrete

Aggregate requirement for asphaltic concrete (AC) is presented here because the source of this material is included in the main research discussions aggregate quality. The source of AC aggregate, 'Ahononou Quarry is one of the quarry sites discussed in Chapters 6 and 7 regarding test results and a future option for aggregate supply.

The grading was based on combined aggregate grading with an introduction of a filler to be 2% of the combined aggregate mass. Filler is to be hydrated lime to comply with AS2357.

Table 5.2. Basecourse grading envelope for asphaltic concrete. (Source: Project for Road Improvement in Tonga, 1994)

Sieve Size (mm)	**Percentage Passing by Mass**
19.0	100
13.2	85-100
9.5	70-85
6.7	62-75
4.72	53-70
2.36	35-52
1.18	24-40
0.600	15-30
0.300	10-24
0.150	7-16
0.075	4-7

Australian standard AS2150 Appendix K was specified for AC aggregate testing. 'Ahononou Quarry was also nominated by the engineer as the project aggregate source.

Specifying the use of filler in the mix is interesting because mineral fillers serve the purpose of improving the stiffness of the mix when the finer than the asphalt film thickness particles combines with asphalt binder forming a mortar or mastic. The slightly larger particles improve the contact point between aggregate particles.

Mineral Fillers as per NZTA TNZ M/10 are finely ground particles of limestone, hydrated lime, Portland cement or other non-plastic mineral matter, predominantly 0.075mm that is added to the mix. This might appear to be a minor issue but in the bigger context, it demonstrates a lack of understanding or knowledge of the limestone aggregate being used for the project.

The combined aggregate specification was based on Australian Standards that is challenging to achieve due to the great disparity between the aggregate materials available in Tonga compared to New Zealand and Australia.

5.3 Integrated Urban Development Sector Project (IUDSP) 2010

Technical Assistance to the Kingdom of Tonga for Preparing the Integrated Urban Development Project was financed by the Japan Special Fund via the Asian Development Bank (ADB): TON 38160 Sept 2005. Government of Tonga requested assistance to improve urban management and living conditions. This was also to address Tonga's Government Strategic Development Plan 7 (SDP7) for 2001-2003 and ADB's country strategy for Tonga.

One of the driving factors for this project was to improve Tongatapu infrastructure in order to meet the growing population demand in the urban areas of Nuku'alofa due to influx of outer island migrants.

5.3.1 Project scope

This part of the overall project included road improvement, chipsealing and road widening, concrete footpath construction, and miscellaneous drainage work including installation of concrete kerb and channels. Project main components were:

- Asphalt concrete - 13 793 m^2
- Chipseal - 82 189 m^2
- Concrete footpath - 19 065 m^2

5.3.2 Contract specification

An introduction of the term coronus material, which was interpreted as the subgrade material, to be tested using Australian standard AS1289.
The specified test method for the coronus material was the Australian Standard AS1289: Methods of testing soils for engineering purposes.

- Moisture content: AS1289.2.1.1 or AS1289.2.1.4 at the Engineer's discretion
- Liquid limit: AS1289 C1.1
- Plastic limit: AS1289 C2.1
- Plastic Index: AS1289 C3.1
- Particle size distribution: AS1289 C6.1
- California Bearing Ratio: AS1289 F1.1 with the material compacted at optimum moisture content to 95% of maximum modified dry density (AS1289.5.2.1) and soaked for 4 days.

5.3.3 Sub-base and basecourse requirements

Material selection and processing of sub-base and basecourse aggregate were to be from an approved source driven by engineer. Selected material is to be free of organic material and clay.

Similar to previous road improvement projects, 1994/95 road upgrade, AS1289 Methods of Testing Soils for Engineering Purposes referred and AS1141 to test sub-basecourse material. The method of aggregate selection and processing shall be approved by the Engineer before full scale production commences. The materials shall comply with the four day soaked CBR tested in accordance with Queensland Main Roads Test Method Q113-B 1983. The particle size distribution test and results to AS1289.CP6.1 (AS1289.3.6.1).

All quality test requirements were to comply with the respective Australian engineering standards. This is to be further discussions below especially regarding the difference in environmental conditions between these two countries.

5.3.4 Surfacing

Aggregate specification for bituminous surfacing, asphaltic concrete and chipsealing, were exactly the same.

The source of material, as per previous clauses, had to be approved by the Engineer. Approval of the source does not signify approval of the material from the source. If the material is proven unsuitable for the purpose then the engineer can still veto the granted approval.

Generally the aggregate should be clean, hard, dry, tough, sound, crushed stone of uniform quality, free from dust, clay, dirt or other deleterious matter and from excess of flat or laminated pieces and shall be of such a nature that when thoroughly coated with the bituminous material proposed for the work, the coating will not peel off upon contact with water.

In addition, Australian standard AS1141.15 was specified for more specific engineering property testing. A particle size distribution table was also provided to ensure tested material comply within the limits of 20mm or the 14mm nominal sizes.

The following requirements are most relevant to the limestone material tested for this work and also by others.

The flakiness index of the aggregate retained on the sieve stated below for the particular nominal size of aggregate shall not exceed 30% when tested in accordance with AS1141.15.

The portion retained on a 9.5mm sieve shall not contain more than 10% by mass of particles in which the ratio of maximum to minimum dimension exceeds 5:1.

Not less than 60% by mass of the particles retained on a 4.75mm sieve shall have at least two fractured faces."

Table 5.3. Particle size distribution of aggregates when tested in accordance with AS1141.11 shall fall within the limits for either 20mm or 14mm nominal size (IUDSP, 2010).

SIEVE SIZE	PERCENTAGE BY MASS PASSING SIEVE AGGREGATE NOMINAL SIZE		
	20 mm	**14 mm**	**10 mm**
25.5 mm	100		
19.0 mm	95 - 100	100	
13.2 mm	0 - 20	95 – 100	100
9.5 mm	0 - 5	0 – 30	95 – 100
6.7 mm	-	0 – 5	0 – 30
4.75 mm	-	-	0 – 5
2.36 mm	0 - 2	0 - 2	0 - 2
Test for Flakiness Index	13.2 mm	9.5 mm	6.7 mm

5.4 Tonga NIP – Contract No. ZTY01-09, 2009

The engineering design standard for the Tonga National Improvement (TNI) Project Contract No. ZTY01-09, signed between the MOW of Tonga and China Civil Engineering Group Co Ltd, was based on Ministry of Transportation of the People's Republic of China as well as the Tonga's MOWs local construction methods.

Discussions are based purely on the MOW English version of the contract description without any cross referencing to the Chinese road design or construction standards.

Chipsealing work is the major aspect of this project so specific references will be made to chipseal aggregate specifications, in other instances specification is lacking so comments will be added in that respect also.

5.4.1 Background

This project was financed by the People's Republic of China's government. The main objective was to reconstruct 65 road sections in Tongatapu, 5 sections in Vava'u, 2 in Ha'apai and 1 in 'Eua. The driving factors are driven by the NSPF 2009/2010 and NIIP 2010 objectives and recommendations. Identified in the reports was the need to improve the access roads connecting agriculture, fisheries and tourism with their respective sources and destination. Theoretically, the economic benefits should filter down to fuel social development therefore improving people's day to day living.

Overall, 10.4kms of what appeared to be asphalt listed as the proposed pavement type on a list of road information schedule.

Interestingly there are no references to chipseal surfacing. Majority of resurfacing work being done during research site visits was chipsealing. A statement made in 2014 during research discussions by MOW engineers that Chinese engineers managing the project have never heard nor used chipsealing prior to arriving in Tonga.

The preamble for subgrade construction needed careful reading in order to interpret and understand it due to its vagueness which is most likely due to the interpretation differences amongst languages. A preamble example when referring to excavation: "For earth and stone excavation, adopt small-scale and short-distance method. Fill and cut by layers to avoid steep slope formation and affecting vehicle passing. In subgrade construction, remove wasted earth in time, at all times guarantee to have a half width subgrade to open to traffic, cutting or burying subgrade at a large scope is prohibited which shall make traffic block" (Contract No. ZTY01-09, 2009).

5.4.2 Aggregate Material

To further highlight the difficulty in working with engineering standards from a different country, some of the material specification for this work was as follows.

"Tongatapu Island is an island country, rich in coral stone and convenient in mining and transportation. Therefore, for this project, adopt coral stone as subgrade filler and the sand and stone material for pavement shall made by rolling hard coral stone. Crushed stone (coral stone): water absorption ratio: 0.66, crushing value: 0.34(%) ~0.4(%). Abrasion value: 0.36(%) and the adhesion with asphalt: 4 grade. However, the crushed stone for engineering must be purchased from specified quarries. Through investigation, the purchasing price is site price (including load unload" (Contract No. ZTY01-09, 2009).

Furthermore, specifying the source of aggregate from Tonga based quarries provide further interpretation challenges. Upon discussing this project with local technical staff and site visit observations, all work carried out under this project was overseen by; 2 senior engineers, 12 engineers and 3 assistant engineers of Chinese origins. The Tongan technical staff's presence was merely to manage the project relationship and Tongan MOW administrative duties.

Clearly Chinese nationals, Chinese Government and other technical experts controlled the work utilising their own standard specifications. This totally contradicts all previous road construction work completed under Australian and New Zealand standards. It is therefore difficult to include this project in discussing comparative technical requirements with AUSTROADS and NZTA standards.

This project demonstrates internationally financed projects continues to create ongoing design and construction quality problems in Tonga.

5.5 Fillers

Specifying fillers for the 2010 project is interesting because none was used in previous projects therefore lesson learnt were not captured. Limestone aggregate have high percentage of fines therefore there doesn't seem to be a need to use fillers. It therefore demonstrate, as mentioned earlier, the lack of limestone aggregate knowledge. The quality of material used as sub-base fill and the quality chipseal surfacing is unknown.

Comments have been presented regarding the Chinese Government funded 2012 project. During research site and sampling visits in 2015 it appeared that the source of chipseal aggregate was from the Malapo Quarry. An aggregate sample was obtained from the Malapo Quarry site and is part of the material tested for this work. However, based on McCotter et al., (2010) tests results the Malapo Quarry aggregate achieved the worst out of all quarries in Tongatapu. Reiterating comments made by MOW engineers in 2014 that Chinese engineers managing the 2012 project had not heard of chipsealing prior to this project. There are doubts whether the appropriate chipseal aggregate tests similar to TNZ M/6 were applied.

Therefore the view established arisen from reviewing the above projects is that all the General and Specific Conditions of Contract for Tonga roads are totally incompatible with local conditions, local technical skills and do not reflect the quality of the limestone aggregate source locally. The standard specifications are based on Australian, New

Zealand, Japanese, Chinese standards or any other country funding these types of projects.

5.6 Discussions

The aggregate specification for the first two projects, 1994/95 and 2010, were identical. Interestingly fifteen years later after the reconstruction improvement projects in 1995, Tonga had not developed its own specification based on previous experience.

Clearly the environmental conditions between Tonga, New Zealand and Australia are all different in general though it could be argued that in some isolated locations in Australia similar conditions to Tonga can be observed.

Pavement constructed in 1994 and 1995 were predominantly asphalt concrete overlay of the existing chipseal surface, minor road widening and associated drainage improvement work. These two contracts were supervised by Japanese engineers whilst the construction work was carried out by a combination of Australian, New Zealand and assistance from local labour force. MOW technical staff managed the project from the Government of Tonga perspective ensuring the work was done within the expected timeframe.

Discussions was carried out with the 1994/95 project site surveyor (Van Heerswyk discussions, 2015) revealed some of difficulties encountered by the contractor during pavement construction. It was discovered early into the project that the locally sourced limestone aggregate did not meet the specified particle size distribution nor any of the strength properties.

The contractor then opted to engage engineers and technical support staff from New Zealand and Australia to carry out the physical works, assisted by Tongan staff from the MOW and locally sourced labour.

A number of adjustments were made on site, which were not part of the specifications, but an innovative approach ensuring the work could proceed and be completed. These included innovating items such as the drainage boxing system to install complicated design system along the waterfront. The construction engineers discovered, by chance observation during unintentional trials of excessive watering then rolling of limestone basecourse, that the material settled similar to concrete. A basecourse treatment process was then developed prior to placement and rolling.

Aggregate stockpiles were saturated with water prior to loading onto site delivery trucks where basecourse was laid using an asphalt paving machine in order to achieve consistent depth. No grading machinery was used. This approach of laying totally saturated limestone aggregate assisted in achieving higher basecourse density values through partial cementation as a result of rolling (Van Heerswyk discussions 2015).

This is an unconventional approach but it demonstrates the general approach of applying "whatever" standard works in order to get the work done (Van Heerswyk discussions 2015).

The Tongan Integrated Sector Development 2010 project was essentially a chipsealing contract, as the chipseal component of the project was approximately five times the asphalt portion, although it wasn't specified nor described as such.

Contractors engaged to do carry out the 2010 project were made up of Chinese nationals and locally engaged contractor.

Chapter 6

AGGREGATE, AGGREGATE TEST METHODS AND TEST BY OTHERS

In this chapter, aggregate test methods directly relating to this research are reviewed with emphasis on how they inform design. The limitations and challenges confronted whilst attempting to obtain aggregate samples is also covered.

Unbound basecourse material specifications, asphalt concrete and chipseal aggregate requirements are discussed in relation to NZTA and AUSTROADS standards.

A literature review of aggregate tests performed by others on Tonga's limestone to highlight and compare results commonalities.

Aggregate standards specified for Tonga contracts are highlighted and compared to test results.

From a road construction perspective, aggregate is the collective term for sand, gravel or crushed stones, and when a binding medium such as bitumen or cement is added, asphaltic concrete is the resultant compound material. In the case of flexible pavement structures, aggregate can be bound or unbound. In this research, the focus is on unbound granular aggregate material.

Natural aggregates are generally extracted from larger rock formations through an open excavation in quarries. Quarrying is the standard method used in Tonga. Extracted rocks are typically reduced to usable sizes by mechanical crushing before being graded to cover a range of specified sizes in order to make them mechanically stable, workable and compactable.

Aggregate form an integral component in manufacturing asphaltic concrete (92 – 96% of mix) and as chips for chipsealing surfacing where it is the first layer in the structure to absorb vehicle loading.

The final decision on material selection is based on not only the structural requirement but also accounts for economics, durability, workability and acquired experience. The standard reference during design is what are the elastic modulus and Poisson's ratio values. Elastic modulus allows the designer to determine how resilient the material is when subjected to stress and strain which in turn have a direct in the pavement's longevity. Elastic modulus would give an indication of how stiff the material is in order for the pavement to resist damage likely to be caused by repetitive loading.

As mentioned earlier, unbound aggregate material is required to resist loading by having adequate compacted compressive and shear strength to alleviate rutting and shear respectively. It is required to attain high compacted modulus for subgrade protection.

It is therefore crucial and necessary to test and confirm the suitability of the limestone as an unbound aggregate material as well as suitability to be used as a surfacing aggregate.

6.1 Aggregate selection

Using New Zealand's TNZ M4 basecourse selection process at the source, usually quarries similar to Tonga situation, durability and material gradation can be determined before full production takes place. Once the source or the quarry is identified, three tests are conducted at the source in order for the material to be accepted for production. They are crushing resistance, weathering quality and CBR test. All are fundamental durability checks.

Four further tests are carried out during production namely sand equivalent, broken face content, clay or plasticity index and particle distribution before the aggregate can be used for pavement construction.

6.1.1 Particle size distribution

Establishing the particle size distribution of a sample of aggregate gives a graphical representation that can be compared to the design specification. The testing can be conducted at the source or it can be done at the production process away from the source. Sampling of material is another criteria to comply with and TNZ M4 specification also include details of the sampling description process.

Gradation is critical because it has a direct influence on the quality and cost of the constructed pavement. The limit placed on the gradation depends on the end use of the product. Dense asphalt or basecourse material (vs sub-base) require high quality material and densely graded. Below is the TNZ M4 particle size distribution envelope limits shown in Table 6.1. This is the requirements for material to be used in New Zealand's state highways and heavily trafficked roads.

Table 6.1. TNZ M4 particle size distribution envelope limits for an individual sample showing both AP40 and AP20 aggregates (Adopted from TNZ, 2006).

Test Sieve Aperture	Maximum and Minimum Allowable Percentage Weight Passing	
	AP40 (Max size 40mm)	AP20 (Max size 20mm)
37.5mm	100	-
19mm	66 - 81	100
9.5mm	43 – 57	55 – 75
4.75mm	28 – 43	33 – 55
2.36mm	19 – 33	22 – 42
1.18mm	12 – 25	14 – 31
600μm	7 – 19	8 – 23
300 μm	3 – 14	5 – 16
150 μm	0 – 10	0 – 12
75 μm	0 - 7	0 - 8

6.1.2 Durability

Durability of the aggregate is considered a crucial property to assess for UGM due to the expected amount of stress they are to withstand during the design life of the pavement. There are a number of available tests to determine aggregate's inherent level of resistance to breaking down.

For this research, aggregate crushing resistance test was chosen because it is one of the tests specified for TNZ M4 and it is also comparable with McCotter et al (2010) results reviewed below.

The test is undertaken by gradually applying a metal plunger of a standard load of 400kN to an aggregate material sample size of between 10 – 14 mm, contained in a test mould. As per the TNZ specification, the amount of material passing 2.36 mm sieve in percentage of the total weight of the sample is referred to as the Aggregate Crushing value (ACV). , TNZ M4 specifies that less than 10% fines passing 2.36 mm sieve size shall be produced when subjected to a load of 130 kN. High percentage fines produced (> 10%) at the end would indicate that the material is weak. Alternatively, the ten percent fines value can be used where the load (kN) that produces 10 percent of fines passing 2.36 mm sieve so a load value is expressed instead of the percentage of fines.

The aggregate must not degrade under the action of traffic hence there is a minimum hardness criteria. A crushing resistance (CR) > 13OkN is specified for the TNZ M/4 basecourse aggregate. Low CR values or > 10% fines, may result in premature pavement degradation due to reduced elasticity and resilient strain, increased permanent strain.

The correlation between material failure and loading due to loss of resilient strain is illustrated below as demonstrated by Werkmeister (2003) when aggregate samples are tested through the repeated triaxial loading (RTL). Increasing load repetitions eventually leads to increase in permanent deformation (permanent strain) which eventually leads to sample failure.

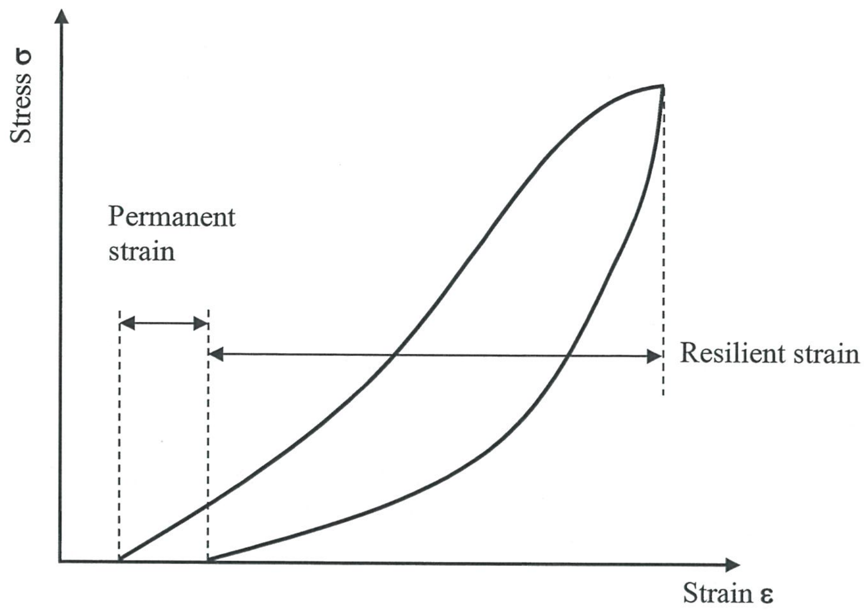


Figure. 6.1. Hysteresis loop for viscous elastic UGM behaviour (Modified from Werkmeister, 2003 and Margan et al., 2014)

As discussed earlier, resilience is the portion of energy imparted into pavement aggregate when a load is applied and then completely regained when the load is removed. It is however a lot more complex than simplified diagram in Figure. 6.1.

6.1.3 Weathering Quality Index

Another useful and applicable to Tonga test is the Weathering Quality Index, the seasonal resistance of the material, measured by the Weathering Quality Index, similar to the Soundness Test. This test is used to assess aggregate resistance to the effects of wetting, drying, heating & cooling expressed in a series of two letter designations based on cleanness value and percentage of material retained on a 4.75mm sieve at the conclusion of the test as shown in Table 6.2.

Table 6.2. TNZ Weathering quality index (Adopted from NZS 4407, 1991).

	Percentage retained on 4.75mm Sieve		
Cleanness Value	**96 to100**	**91 to 95**	**Up to 90**
91-100	AA	BA	CA
71-90	AB	BB	CB
up to 70	AC	BC	CC

M/4 requires an AA, AB, AC, and BA, BB or BC designations for basecourse. M/6 only allows AA or BA to be used for sealing chip. Therefore there's a lot more flexibility as a sub-base or basecourse material but not for chipseal surfacing where the highest level of stress from loading is being absorbed by the pavement.

The aggregate must also not degrade under the action of environmental changes and hence must meet minimum chemical weathering criteria. This is tested using the breakdown after soaking in a solution of Sodium Sulphate. TNZ uses a qualitative weathering index for acceptance. A low grade weathering resistance may cause premature attrition degradation breakdown of the aggregate and the generation of plastic fines.

Some of this rock can be porous, expressed in saturation moisture content, where soft chalk like rocks can be as high as 25 – 30% and hard limestone can be as low as < 1%.

Table 6.3 is a summary of TNZM4 desirable properties. (Pidwerbesky, 2015).

Property	Test Method	Criteria
Crushing Resistance	NZS 4407 Test 3.10	130 kN load produces < 10 % fines
Weathering Index	NZS 4407 Test 3.11	AA, BA, AC, BB, CA
California Bearing Ratio	NZS 4407 Test 3.15	≥ 80
Broken Faces	NZS 4407 Test 3.14	≥ 70% with ≥ 2 broken faces
Plasticity Index	NZS 4407 Test 3.04	≤ 5
Maximum Particle Size		AP40 – 40mm AP20 – 20mm

6.2 Aggregate fines and water

Lekarp (1997) discovered that UGM resilient behaviour is affected by factors such stress conditions being subjected to, density, moisture, fines, grading, and aggregate type, number of load applications, stress history, load duration, frequency and sequence.

An important factor to consider is the fine contents of the aggregate determined from the particle distribution test. Werkmesiter (2003) stated that excessive amount of fines in the aggregate mix has a negative impact on pavement stiffness and permanent deformation.

In dry conditions, dry density value is relatively high when the level of fines is also high. However, with high levels of fines, and introduction of moisture, the moisture content of the material becomes the important overriding factor. Resilient modules decreases as well as shear strength with high levels of fines, but is also moisture content dependant.

Araya (2011) reiterated the work by Sweere (1990), that the effect of water on the behaviour of granular materials is greatly related to the amount and nature of the fines.

The correlation drawn by Araya (2011) is that moisture affects fine graded material with excessive amount of fines to be much greater than coarse graded crushed rock.

A certain amount of water is required for aggregates in order to achieving maximum dry density, referred to as optimum moisture content. Either side of this value would not achieve maximum value. More detrimental to the aggregate is when moisture is too high. When water saturates a pavement layer with excessive amount of fines, some of the fines are washed out replaced with water. Vehicle loading will increase the pore pressure and level of saturation. This leads to a number of problems referred to by Margan et al., (2014). Pore pressure within the aggregate mix is increased which decreases the effective stress which would then destabilise the mix. Permanent resilient strain increases which leads to deformation as mimicked by the RTL test.

In well compacted granular material under stress with appropriate moisture content, there are numerous contact points amongst granular particles, keeping the structure stable. If the moisture content increases above the optimum level, and the transmitted load is transmitted equally in all directions, Pidwerbesky (2015) argued that the particles are then forced apart therefore the structure becomes vulnerable to deformation.

The level of fines and moisture effects on pavement structures are some of the crucial points to be considered in Tonga's situation. Tests carried out by others and also tests done for this research would show that the portion of material from the particle size distribution data passing through the 2.36mm sieve, regarded as fines in TNZM4, is excessively higher than expected.

6.3 Tests by others

There have been attempts in the past, prior to this research, by various internationally funded consultants including the 1991 MOW laboratory funded by The Overseas Service Bureau of Australia, to try and establish an overall understanding of the locally sourced limestone aggregate. Following is a review of the tests carried out by these consultants which covers aspects of the limestone properties not tested for this research.

It is important to get an overall view of what has been established prior to this research filling in the gas and maximise the approach taking into account this research's financial limitations. Costs of importing aggregate for testing in NZ for instance prevented the importation of enough volume of aggregate to conduct all possible tests. Also by reviewing previous work revealed and highlighted an overall lack of effort in ensuring the appropriate aggregate used.

Therefore there had been considerable widespread total pavement failure occurring throughout Tongatapu before the 1990s was still being encountered prior to the commencement of this work in 2014. This would suggest major flaws or systematic failure exists within the ministries responsible for the road infrastructure and all the international consultants that were in charge of designing, building and managing road construction projects over the years. This leads on to previous aggregate testing efforts conducted in the past.

6.3.1 Candler (1992)

Candler (1992) was an Overseas Service of Australia technical volunteer working for the MOW in 1989. He was in charge and ran the MOW laboratory during his time in Tonga.

Candler (1992) worked on concrete production testing aggregate and the test was based on AS1141, the same aggregate standard tests specified for the 1994/95 roading projects. Any references or correlation was made regarding road construction is of interest to this report. Candler (1991) does set the acquired knowledge background existed at the before the 1994-1995 Japan funded projects.

Aggregate was quarried, crushed and then screened into various sizes and the main characteristics that were of significance to Candler (1991) were cleanliness of screened product, density of the material and the water absorption capability of the aggregate. It is noted that the cleanliness or quantity of fine material in samples was excessive.

Table 6.4. Test results from a number of Tongatapu quarries showing dry bulk (specific gravity) densities and water absorption (modified from Table 2. Candler, 1991)

QUARRY	BULD DENSITY (dry) t/m^3	WATER ABSORPTION (%)
'Ahononou	2.49	2.6
Royco	2.27	4.9
Pili	2.18	5.4
Fualu	2.14	6.2
Fualu (Talofolo)	2.13	7.8
Longoteme	2.07	7.8
Talafo'ou	2.04	10.0
Mataki'eua	2.04	7.7
Malapo South	1.84	12.9

Summarising the general comments made with regards to the results in Table 6.4.

1. The densest aggregate in Tonga did not meet the standard specification used elsewhere in the world.
2. Due to the wide variety in density values, the samples were divided into a group of four.

- 'Ahononou quarry which contained the densest material
- Medium – High Density 2.20 – 2.40 t/m^3
- Medium – Low Density 2.00 – 2.20 t/m^3
- Malapo South and another quarry with the lowest density values

Due to the obvious weakness in density of Tonga's aggregate, it was widely expected that high quality strong products such as concrete couldn't be achieved. The lack of technical staff and poor laboratory equipment contributed to this due to limitations in MOW's ability to conduct rigorous research work.

Fletcher Construction using aggregate from 'Ahononou, plus some technical innovative work referred earlier, were able to produce 40 MPa concrete previously deemed unachievable by local engineers.

There is a strong correlation between low density and high water absorption which is a major challenge for both concrete and AC pavement production which then lead into some major technical challenges during construction. Water absorption and density issues can be addressed with the water-aggregate-cement mix ratio during concrete production. This isn't as straightforward in road construction due the long term effect excess water can have on pavement structures. Having excessive water absorbed by aggregate particles would impact on the adhesion of bituminous products to the aggregate. This would be a problem during AC production.

Therefore to conclude, by the end of 1991, the density of limestone rock was known, cleanliness and water absorption problems were highlighted by Candler (1991) at the MOW.

6.3.2 Harrison (1993)

Funded by the Overseas Development Administration (ODA) under the ODA/British Geological Survey (BGS) research development programme, part of the British Government's contribution toward developing countries, Harrison (1993) set out, after a previous Tonga and Fiji reconnaissance visit in 1991 to assess the sand resources, limestone aggregate evaluation, and minerals planning guidance for Tonga.

The focus of the Harrison (1993) report was the evaluation of limestone resources and the quarrying industry at the time. Field investigations were carried out in October 1992 and laboratory assessments undertaken between November 1992 and February 1993.

The overall objective was to assess and analyse Tonga's limestone physical and mechanical properties, complemented by petrographic descriptions together with some detailed chemical analysis. These results, conclusions and recommendations in principle were then applied to other Pacific islands similar to Tonga.

There were seven active quarries operating in Tongatapu at the time with a further seven disused or abandoned. The size of the quarries varied from small, < 1 hectare, to more extensive ones, > 2.5 hectares.

The excavated depth of active quarries varied between shallow single bench operations (4 – 7m high), to some much deeper sites (10 – 30m high). Many of these quarries are excavated all the way down to the water-table which puts the underground water supply at risk of contamination.

Limestone production quantities were relatively high during the period from 1988 to 1991 that included an extra 133,400 m^3 for the foreshore construction project along the waterfront in Nuku'alofa and reconstruction work at Fua'amotu airport.

Table 6.5. Limestone production in Tonga from 1988 to 1991. (Source: Harrison, 1993).

Production (m^3)	**1988**	**1989**	**1990**	**1991**
Government Quarries	59,225	42,237	36,574	54,583
Private Quarries		76,027	13,087	15,006
TOTAL	> 59,225	245,664	65,661	69,589

Notes:
Total for 1988 does not include production from private quarries.
Total for 1989 includes 127,400 m^3 for construction of Nuku'alofa foreshore
Total for 1990 includes 16.000 m^3 for Fua'amotu airport construction

It appeared that minimal progress had been achieved since Harrison (1993) report based on the observations made during site visits for this project in 2014.

The report also confirmed MOW's 'Ahononou Quarry as being the only supplier capable of producing chipsealing chip due to it being the most dense therefore less porous, lowest water absorption as per Candler (1992).

All limestone aggregate testing were done by BGS at their own testing facility in Keyworth England. Samples were taken from working or disused quarries, were unweathered or partially weathered, either from the quarry face or stockpiles

X-ray fluorescence spectrometry (XRF) was used for chemical analysis, combined with both field and laboratory lithology observations, to give the chemical quality as well as the lime (CaO) quality assessment.

The BGS team used British and ASTM standards to measure both the physical (density and porosity) and mechanical (aggregate response to external stimuli, impact or shear stresses). Table 6.6 are examples of tests used.

Table 6.6. Typical tests used by the BGS team to assess limestone properties (Harrison, 1993).

Physical Tests:	
Flakiness index	Measure degree of flaky particles
Relative density	Specific gravity
Water absorption	Measure porosity and capacity to absorb water
Mechanical Tests:	
Aggregate impact value	Measure resistance to granulation under impact stress
Aggregate crushing value	Measure resistance to crushing a gradually applied load
Los Angeles abrasion value	Measures resistance to attrition by impact and abrasion forces

The results from the tests were to comply with the typical values for roading aggregates set out in Table 6.7.

Table 6.7. Typical requirements for pavement aggregates as tested by the BGS team 1993 (Source: Harrison, 1993).

Test	**Description**	**Value**
Particle density	generally	>2.65
Water absorption		<2%
Flakiness index		<35
Aggregate impact value (AIV)	generally	<25
Aggregate crushing value (ACV)	generally	<30
Los Angeles abrasion value (LAAV)	generally	<40

Following are the results from tests carried out by Harrison including some of the results from Candler (1992) work for comparison purposes and for ease of reading.

The BGS team results were achieved using the BS812, 1990 and the MOW Candler 1992 results were achieved using AS1141 Sections 5 & 6; Bulk Density and Water Absorption.

Table 6.8 shows the physical properties of limestone (modified from Harrison 1992 Table 5).

Quarries	Flakiness	Relative Density		Water Absorption (%)	
	BGS	BGS	MOW Candler (1991)	BGS	MOW Candler (1991)
'Ahononou	14	2.44	2.49	2.1	2.6
Farm	17	2.35		3.1	
Pili	5	2.22	2.18	4.9	5.4
Longoteme	5	2.10	2.07	7.5	7.8
Tapuhia	18	2.17		5.6	
Fualu	9	2.09	2.14	8.0	7.8
Cockers	16	2.32	2.27	3.9	4.9
Mataki'eua	8	2.06	2.04	9.9	7.7
South Malapo	9	2.14	1.84	8.1	12.9

The following are Tongatapu's limestone aggregate mechanical properties as tested by BGS based on the American Standard tests for aggregate strength and durability.

Table 6.9. Mechanical properties of Tongatapu's limestone aggregates (modified from Harrison, 1993)

Quarry	**AIV**	**ACV**	**LAAV**
'Ahononou	27	31	26
Farm	31		29
Pili	34	37	35
Longoteme	37	40	38
Tapuhia	37	40	37
Fualu	44	45	38
Cockers	38		39
Mataki'eua	48		38
South Malapo	42		39

Note: *AIV* (aggregate impact value) and *ACV* (aggregate crushing value) are based British Standards aggregate strength tests.
LAAV (Los Angeles Abrasion Value) is the American Standard for strength and durability tests

AIV – general requirement <25, all results failed to meet this.
ACV – general requirement <30, all results failed
LAAV – All passed although 7 out of the 9 quarries had values ranging from 35 – 39.

As a result of the above tests, three Groups (Table 6.9) were created to indicate where the best source of aggregate can be sourced from. Group 1 rated as the best source and can be used for a variety of construction purposes including pavement sub-base, basecourse and surfacing.

Group 2 have moderate strength, medium low density and more porous aggregate than Group 1. Densities range between 2.10 and 2.33 with water absorption values of between 3.9% – 7.5% therefore they are much weaker than standard specifications elsewhere. Group 3 was the weakest of the three groups rendering it only suitable for construction fill or sub-base material.

Table 6.10. Three classification groups based on durability and water absorption properties. The quarry sources are the Tongatapu based operations only (Source: modified from Harrison, 1993).

Group	Possible Use	Description	Typical Index Test Value	Quarry Sources
1	Road surface aggregate, also suitable for construction purposes	Relatively strong % durable. Least porous aggregate	AIV & LAAV < 32, Rel. Den >2.3, Water Abs. < 4.4%	'Ahononou, Farm
2	Base & sub-base	Moderate strength & durability. Porous aggregate	AIV & LAAV < 40, Rel. Den >2.1, Water Abs. < 8.0%	Pili, Longoteme, Tapuhia, Cockers
3	Suitable for road sub-base or construction fill	Weak with low durability. Highly porous, low density	AIV & LAAV > 40, Rel. Den <2.1, Water Abs. > 8.0%	Mataki'eua, South Malapo

Harrison concluded that the soft nature of the limestones results in high percentages of very fine material produced during the extraction and crushing processes. Aggregates are contaminated at quarries due to lack of adequate processing systems and poor control of overburden soils.

There are blocks of 'hard' limestones that are usually left out of the crushing process because they are too big or too strong the crushing machines so they are left behind and become quarry waste. There's a wide variation of aggregate quality in many properties throughout Tongatapu and their locations determines the quality. Majority of the limestone aggregate does not meet international standards for pavement construction.

The palaeoreef limestone from 'Ahononou and Farm quarries located at the southern end of Tongatapu are the best sources of aggregate in Tongatapu as per the above results.

6.3.3 McCotter et al., (2011)

This project was borne as a result of the Tongan Government seeking assistance from the Pacific Region Infrastructure Facility (PRIF) as an attempt to improve its road maintenance in a sustainable manner. The focus was to establish a cost effective model providing materials and equipment to maintain the road infrastructure by the private sector.

This review will focus mainly on the limestone engineering properties aspects. It is accepted that the demand on the limestone resources is strong and continuous. Several projects were being realized or concluded at the commencement of this research.

As part of the methodology for the institutional assessment report, after reviewing historical reports by Candler (1992) and Harrison (1993), rock aggregate samples were acquired for re-testing from seven quarries, six of which were privately owned and the MOW 'Ahononou quarry.

Fifty kilogram samples of crushed sealing aggregates collected from sealing aggregate stockpiles were arranged into 25kg bags suitable for air transport.

The Government of Tonga and DHL Couriers air freighted the samples to Sydney and to the Boral materials testing at Baulkham Hills ISO/IEC Standard 17025 accredited laboratory. The samples were tested according to Australian Standard AS1141 – Methods for Sampling and Testing Aggregates (McCotter et al, 2010), previously specified for Tonga's road construction projects.

AS1289 had also been specified as basecourse material standard reference for previous projects in 1994, 1995 and 2010. Particle size distribution describe Table 6.10

Table 6.11. Sealing aggregate particle distribution results from the 7 selected quarries (modified from McCotter et al, 2010)

	Quarries							
Sieve Size (mm)	**Chinese**	**Malapo**	**Ahononou**	**Nishi**	**Royco**	**Tafolo**	**Vete**	**Specification AS1141**
26.5						100	100	
19	100	100	100	100	100	90	97	**100**
13.2	57	65	56	61	62	63	41	**90-100**
9.5	6	20	17	8	30	37	11	**0-30**
6.7	5	6	6	5	11	24	8	**0-5**
4.75	5	5	4	5	6	19	8	
2.36	5	5	4	5	6	17	8	
1.18	4	5	3	5	5	16	7	**0-1**
< 75μm	2	2	2	2	5	5	3	**≤ 1**
Ratio 2:1	5	4	10	2	4	5	2	**≤ 35**
Ratio 3:1	0	0	1	0	0	0	0	**≤ 10**

This material was assessed whether it was suitable as chipsealing or asphaltic concrete purposes. The report highlighted how easily it was for the samples to be broken down. None met the retention specifications at 13.2mm sieve, some met the 9.5mm and the 6.7mm requirements and overall there's high percentages of finer material in 4.75mm - 75μm range were present. This indicated that the limestone was relatively weak and it crumbles under loading and produces high level of fine material as a result.

When measured against the TNZ AP40 specification, which has lower specification where aggregate is subjected to a much lower load compared to the chipseal aggregate requirements, the same conclusions can be drawn. All aggregate materials tested does not comply with the TNZ AP40 or M4 specification nor the TNZ M6 for chipseal chips.

As discussed in earlier chapters, chipseal surfacing makes up the majority of Tongatapu's road infrastructure, hence this report could have been more directed towards solutions instead of softening the approach to fit the results, as mentioned by McCotter et al (2010).

In the report it is suggested that the only method of cleaning out the finer particles is through wet sieving which isn't being done in Tonga due to lack of equipment. It does result in higher bitumen spray rate but the main problem is the hardness of the aggregate, highlighted by the low crushing resistance values below and the wet strength, refer Table 6.11.

McCotter et al., (2010) raised a proposal of how to accommodate and accept the obvious crushing value weakness of the aggregate was use an approach previously adopted by Gold Coast Council in Australia.

"If suitable igneous aggregates are available, a normal specification limit is not more than 20%, when tested against Australian Standard AS1141.21 (Gold Coast City Council, 2005). However, if highest quality material is not available, this value is usually relaxed to be as high as 35% (Waltham, 1993). First Highway Consultants Co Limited, the design consultants for the Tonga National Road Improvement Project, adopted this more liberal upper limit of 35% as the maximum that should be used" (McCotter et al., 2010).

The results clearly show aggregate crushing value failures in all quarries sites, ranging from 50.2 % to 36.4%. 'Ahononou quarry being the closest to the 35% upper limit mark. If the proposed relaxing of crushing value is adopted making it 35%, the overall outcome is still a 100% failure for all quarries and 'Ahononou is the closest at 36.4%. Also worth noting that this would be a 100% failure to meet the < 10% TNZ M4 crushing value requirement.

When assessing the wet strength specification of ≥ 150 kN all quarry sites failed.

It was concluded that their test results were consistent with work done by Harrison (1993) and Candler (1992), reviewed in this chapter. 'Ahononou quarry achieved the best results.

Table 6.12. Sealing aggregate ALD, ACV, Wet Strength and Wet/Dry results from the 7 selected quarries (modified from McCotter et al, 2010)

	Quarries							
	Chinese	**Malapo**	**Ahononou**	**Nishi**	**Royco**	**Tafolo**	**Vete**	**Specification AS1141**
Average Least Dimension	10.7	9.2	8.4	10.6	8.2	8.4	11.3	≥ 7
Crushing Value (%)	43.3	41.3	36.4	42.7	46.4	50.2	43.4	35
Wet Strength (kN)	75	85	126	89	79	45	76	≥ 150
Wet/Dry Variation	10	11	5	12	9	26	11	≤ 35

Chapter 7

TEST RESULTS, DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

Observations of Tonga's road condition were carried out in 2013 in order to understand the extent of the pavement problems. Following the road inspections, site visits to the MOW premises and discussions with technical staff, the common theme raised as the probable pavement disintegration pointed toward the limestone aggregate. Initially basecourse material was identified as the main target.

Upon returning to Auckland to seek advice from laboratory technical staff at OPUS International, it was recommended in order to achieve the proposed research objectives, four properties of the limestone aggregate were to be assessed for this research.

The next step was to seek permission for the research work from Tonga's Ministry of Education, and this process took three months. On top of the education ministry's requirement, the Minister of the Environment, Hon. Ma'afu, was also required to approve extraction of aggregate for testing.

At the next level were the owners of the quarries which included another noble, Hon. Nuku who gave the approval to obtain samples from the Kolonga Quarry.

7.1 Tests for this research

Table 7.1. Aggregate test objectives and methodology presented as part of the proposal for this research work

Aggregate Test Objectives	Methodology
Determine the Particle Size Distribution envelope for Tonga's limestone	NZS 4407: 1991, Test 3.8.1 Wet Sieving Test.
Determine the crushing resistance of aggregate samples from existing sources	TNZ 4407: Test 3.10
Determine Weathering quality	TNZ 4407: Test 3.11
Determine the California Bearing Capacity (CBR) - compared with Queensland Road methods	NZS 4402:1986, Test 4.1.3, NZS 4407:1991, Test 3.15 Test Methods Q113-B 1993.

7.1.1 OPUS International Consultants Ltd (OPUS)

Auckland University of Technology did not have the facilities to test aggregates for this purpose. OPUS was approached to carry out the testing due to their vast experience and knowledge of aggregate from the Pacific from previous projects. They are also an IANZ Accredited laboratory together with the facility to get MAF approval for importation of aggregate samples from Tonga.

OPUS have an ongoing business relationship with DHL the international courier company which greatly assisted in transporting the aggregate samples from Tonga to Auckland.

All site visits to Tonga were funded outside the University as well as costs to import samples and the laboratory tests. Cost implication in turn limited the volume of imported material for testing therefore this is reflected in the number of completed tests.

7.1.2 Reasons for selecting the tests

Due to limitations referred above, particular the volume of imported aggregate, the most effective tests were decided to cover:

- TNZM4 Specification for basecourse aggregate – Item 3, 3.3 Source property tests, 3.3.1 to 3.3.3 which can determine at the outset whether the material is strong enough to be advanced to the production test phase, TNZM4 – Figure 1.
- TNZM6 Specification for sealing chip – Item 3, 3.2 and 3.3, and
- Asphalt concrete aggregate requirements TNZM/10 Item 4. Materials, 4.2 Coarse aggregate Table 4.1 and Item 5. Mix design, MIX15 Table 5.1 in TNZM/10.

7.1.3 Testing process

Samples were collected from three quarries that gave permission and made aggregate available for testing. For confidentiality and commercial reasons, quarry owners did not want to be identified nor the exact sample locations except for the Pili Quarry, located in Nuku'alofa.

Sampling was carried out by the researcher aided by a colleague who is a civil engineer with 30 years of work experience in geotechnical investigation and materials testing. Sample bags were filled with careful consideration that a true representation of the overall source was achieved and the TNZM/4:2006 Notes to the specification for basecourse aggregate sample sizes requirement for the chosen test methods refer Table 6.13. The bagged samples were then transferred to the OPUS material laboratory in Albany, New Zealand via DHL Tonga.

Fig. 7.1. Samples storage and then being mixed for sieving at OPUS laboratory, Albany, Auckland.

All tests were conducted at the OPUS laboratory by the writer and an OPUS laboratory assistant under the supervision of the Assistant Laboratory Manager, David Hotham. David has been involved with testing geotechnical material for over 30 years. He has vast experience and knowledge of testing aggregate in New Zealand and the Pacific Islands which was one of the reasons for choosing OPUS laboratory for conducting the analysis.

7.1.4 NZS 4407: 1991, Test 3.8.2 Particle Size Distribution envelope for Tonga's limestone

The test is to follow the requirements set out in TNZM/4, 4.2.3 and the range of result to be measured against the limits envelope in Table 2.

The formula for gradation curve is:

$$P_n = \frac{M_n}{M} \times 100$$

Where: M_n = *mass of soil retained on sieve 'n'*
M = *total mass of sample*

Here are some of the sieved samples.

(a) (b) (c)

Fig. 7.2 (a) Aggregate retained on the 13.2mm sieve, (b) 9.5mm sieve and (c) 4.75mm sieve

7.1.5 **TNZ 4407: Test 3.10** - Determine the ***crushing resistance*** of aggregate samples from existing sources

The test consists of placing the specimen of aggregate in standard steel mould and inserting a steel plunger into the mould on top of the aggregate. The sample is then subject to a compression test under standard load, 130 kN for TNZM/4 (4 tonnes per minute applied until 40 tonnes is reached). The aggregate is removed from the cylinder (already weighed as W_1) is sieved through the specified 2.36mm sieve.

The crushing value is defined as the ratio of the weight of fines passing the 2.36mm sieve (W_2) to the total weight (W_1).

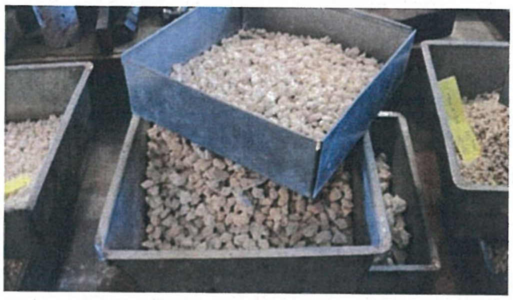
$$ACV = \left(\frac{W_2}{W_1}\right)100$$

Where: W_1 = total weight of sample
W_2 = weight of material passing 2.36mm sieve

Note that TNZ M/4 specified less than 10% fines passing the 2.36mm sieve should be produced.

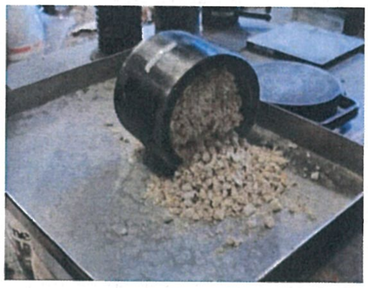
The laboratory apparatus used for the crushing vlaue testing is illustrated below.

(a)



(b)

(c)



(d)

Fig. 7.3 (a) Samples separated into aggregate sizes. (b) Test cylinder is ready and the plunger placed on top ready to be placed on the testing machine. (c) Sample being loaded, maximum load of 40 tonnes. (d) Tested cylinder removed from the testing machine ready for the samples to be removed and sieved.

7.1.6 TNZ 4407: Test 3.11 - Determine Weathering Quality Index

It was requested to apply this test to all quarry samples provided. Due to the lack of sample material and the original approach was to test material for the purposes of being used as basecourse, it was only possible to test a batch supplied for Quarry 2.

This was also done in order to have some comparative view of previous tests done by McCotter et al (2010), reviewed above, for chipsealing aggregate in Tonga.

7.1.7 NZS 4402:1986, Test 4.1.3, NZS 4407:1991, Test 3.15

Test Methods Q113-B 1993. - Determine the California Bearing Capacity (CBR) - compared with Queensland Road methods

The CBR test was not possible to conduct due to lack of sample quantity in the end. Restrictions and barrier to this work was raised earlier and it will form part of the discussions in Chapter 7.

7.1.8 Test Results

It must be noted that access to the laboratory was limited due to the fact that OICL operate as a commercial organisation. The work requested for this report was carried out on a commercial basis which had an impact on any ability to manipulate tests and data prior to presentation of this report, unlike educational institution laboratory facilities.

Access was allowed to the laboratory in Albany to discuss the testing process and requirements for this research work. Viewing samples being delivered, prepared and readied for testing was also permitted.

Advice was received from the laboratory technical staff on what was appropriate and achievable was accepted as part of the test arrangements.

The financial contribution from this research also dictated the number of tests that was possible to be conducted.

7.1.8.1 Crushing Resistance

Four Crushing Resistance tests were requested to be carried out and results were given back based on what OPUS present to commercial clients. This comprised of the paperwork included in APPENDIX 6.

Table 7.2. Crushing resistance test results from OPUS International Consultants Ltd laboratory (OICL).

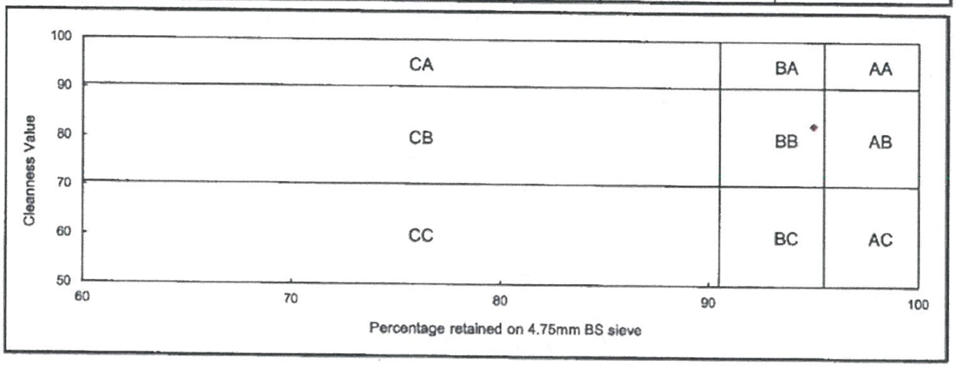
Quarry	Crushing Resistance	TNZ M/10 Basecourse	TNZ M/6 Chipseal
1	< 130 kN 17.1% fines	Test load 130kN < 10% fines	Test load 230kN < 10% fines
2	< 130 kN 18.1 fines	Test load 130kN < 10% fines	Test load 230kN < 10% fines
3	< 130 kN 20.9 fines	Test load 130kN < 10% fines	Test load 230kN < 10% fines

7.1.8.2 Weathering Quality Index

Percentage material retained on 4.75mm BS Sieve after 10 cycles:	95
Cleanness value after 10 cycles:	83
Weathering Quality Index (refer Table 7.3):	**BB**

Table 7.3. Overall results for the Weathering Quality Index from OICL.

Cleanness Value	Percentage Retained on 4.75mm Sieve 96 - 100	91 - 95	up to 90	Specified
91 - 100 71 - 90 up to 70	AA AB AC	BA BB BC	CA CB CC	NZTA M/6:2011 AA, BA



7.1.8.3 Particle Size Distribution

Due to the lack of sample material and as advised by OICL laboratory staff it was only possible to carry out a representative analysis of the particle size distribution for all 3 quarries.

It was also important to have some data, however limited it was, to compare with and confirm work done by others in the past.

Rock is crushed to give a specified grading. The grading has a strong influence on the behaviour of the material. For roadbase or unbound structural layers the grading is more closely specified and is continuous (ie TNZ M/4.) For subbase the specification can be relaxed. (TNZ M/5) TNZ requires a dense grading curve with few fines (<5% passing 75µm size) for an M/4 basecourse aggregate.

A poor grading may initiate high attrition degradation breakdown of the aggregate (from excess "rock to rock" stresses) and premature densification and rutting in addition to the possible generation of plastic fines and premature shear failure.

7.2 Discussion

This thesis endeavoured to assess Tonga's limestone engineering properties and whether its performance as a road construction aggregate measures up to contract specifications. The second objective was to assess long term environmental problems relating to limestone quarrying and is there a long term solution for Tonga.

The Kingdom's reliance on international donor to fund projects coupled with foreign technical assistance has impeded any local motivation to standardise its approach to road construction. There are currently no local standards for pavement design or road construction.

Some of the contract technical specifications are similar but overall there is inconsistency and sometimes contradiction that exists. The use of non-applicable, incorrect and inappropriate aggregate specifications for road construction is the main problem and reason for continuous pavement failure over the years. Hundreds of millions of aid donor money has been spent on projects over the last three decades but not much has been done to address the unacceptable rate of pavement deterioration.

Internal migration from the outer islands to Tongatapu the mainland has exacerbated and intensified the overpopulation issues on the capital Nuku'alofa. The number of vehicles has increased rapidly as a result therefore traffic congestion in and around the CBD is worsening. This also contributes to more traffic load and pressure on pavement structures.

The shared responsibilities approach amongst ministries has created a fragmented nature to the overall control of all aspects of the roading infrastructure. There is no consistency for example in the database record keeping over the years. No single report that can be relied upon as the most up to date version of the changes in the nature of road surfacing or the quantity of public roads versus private roads. There has been a total lack of effectively planning and managing maintenance programmes so the need is driven by visual pavement deterioration until total failures is reached before assistance to repair and rebuild is sought.

International consultants have carried out materials testing since the late 1980s and early 1990s to trying to highlight the geological limitations of the local aggregate resource. There hasn't been any work directly relating to the use of limestone aggregate as pavement material until McCotter et al (2010). This work was however an initiative to establish the framework for better maintenance and to encourage private sector enterprises involvement in carrying out infrastructure projects. The testing of limestone aggregate appeared to have been added on in order to "understand the quality of offering from each of the current quarries". Chipseal aggregate quality was only recognized as small contributing factor to pavement design life: "As road longevity depends *partly* on the quality of sealing aggregate" (McCotter et al, 2010).

Over the years 'Ahononou Quarry the best aggregate material in Tongatapu, but it still failed to meet the international standards specified in previous roading projects, is currently inactive and was operated by the Government's Ministry of Works. Land tenure issues are sighted as one of the problems which lead to its closure. Another barrier is the fact that it has reached the underground water level therefore it cannot go any deeper.

There is a link between the best sites for sourcing good quality limestone aggregate and the oldest palaeoreef structure highlighted by Roy 1990. Tongatapu on the most southern point of Tongatapu Island's topography is tilted from the south sloping downwards toward the capital Nuku'alofa. The palaeoreef corals are well cemented with patches of less friable material. Patch reefs are found more towards the north, elevation is lower and the limestone is more porous and softer.

It is relatively obvious by observing the active quarry sites, the palaeoreef location and test results by others and from this research that the quality of material currently being used is of very poor quality. The South Malapo and Malapo sites achieved the worst quality test results and yet this is the site where the Chinese contractors are sourcing their material from in carrying out the most recent reconstruction work.

This raised the question, and is highlighted in this research as part of the ongoing problem in Tonga, of what standard tests are being used by the Chinese contractors in order to get the permission to use the material? Is it merely the fact that all senior engineering staff and project managers are their own therefore there is a contradiction to the contractual agreement? The project is funded by the Chinese Government therefore they are effectively controlling the overall project even though it is a partnership with Tonga's MOW.

Long term environmental degradation issues have arisen as a result of the continuous demand on limestone aggregate. Quarry operations are privately owned. Tonga's land tenure system complicates efforts to apply proper regulation in sustainably managing these operations. No rehabilitation work is required from quarry operators even though there is a requirement for an EIA report for construction operations. There is nothing in the EIA process nor the existing environmental legislation that addresses environmental damage directly associated with limestone quarrying.

Tonga's road infrastructure is one hundred percent made up of flexible pavement. Flexible pavements are basically made up of three structural layers; sub-base, basecourse and surfacing. There are variations to this such as bound sub-base material where addition of stabilisation agents such as cement or lime.

In the 1994 and 1995 Japanese funded projects, the innovative approach described by one of the project engineering surveyors whereby limestone aggregate material intended to be used as basecourse was completely soaked in water prior to being paver-laid. This resulted in a form of rigid finished basecourse surface, although it wasn't specified in the contract, which passed the compaction and relevant tests prior to surfacing.

The 1994/1995 project is viewed by this researcher as one of Tonga's most successful pavement reconstruction projects. It was essentially an asphalt overlay contract with a chipseal membrane on existing failed chipsealed surfacing. In addition extensive associated drainage facilities were installed along Nuku'alofa's waterfront and some of the By-Pass Road sections of the work. There were culverts added to the section heading towards the international airport which by enlarge drained any excessive water likely build up on the southern side of the road. This remains the longest asphaltic concrete road in Tongatapu.

The Main and Trunk Roads 2012 surfacing data shows that there were 287kms of chipsealed and 21kms of asphalt concrete surfacing in Tongatapu. This equates to 93% of roads are chipsealed and asphalt concrete makes up only 7%. Clearly, a large proportion of the network is made up of chipseal surfacing.

Basic traffic and pavement design principles were reviewed in this research in order to make a fairer comparison with Tonga's infrastructure in view of contract specifications as well as international influence. Tonga's infrastructure from pavement and traffic engineering perspective is almost at infancy level. There are vast gaps and contradictory principles to its overall approach. This should be taken into consideration by international consultants responsible for the design and construction over the years.

It is well known, accepted and fully incorporated into pavement designs worldwide, the effect of moisture or excessive amount of water on the longevity or structural integrity of pavements, particularly flexible pavements with unbound granular layers (UGL). Therefore drainage systems are an integral part of any road infrastructure projects. In Tonga however, not much consideration is given to this and the country does not have a public stormwater drainage system. During the 'wet season' which runs from the end of December through to March/April, currently it is a lot more variable due to global warming effects, water is a major problem. Flooding is a common occurrence in low lying areas around Nuku'alofa.

On observing the re-construction/resurfacing of roads managed by the Chinese contractors and the 2010 National Improvement Project, kerb and channels were installed adjacent to newly built footpaths. Shallow cesspits were also constructed with outlet pipes running into private properties, the ocean or into Fanag'uta Lagoon. The end result is private properties being flooded and the receiving waters totally contaminated with untreated surface water runoff. The flooded roads are obviously being subjected to traffic loading and excess water affecting the pavement structure similar to a 'washing machine'.

The basis for road design in Tonga isn't very clear but it is assumed empirically based after discussions with local engineering staff. Empirical in the sense that no mechanistic data is used and work is relied on "local standards" as specified by the Chinese engineers. There are however a lack of database of historical values and methodologies from previous projects for the empirical approach to be used reliably. A mechanistic based design would be better suited.

There are many design approaches available such as the AUSTROADS, NZTA, AASHTO or the SAMDM that could help determine a better solution than the current guesswork approach.

It is essential for future improvements to incorporate an internationally accepted design approach and a testing regime to determine design components such as: subgrade CBR values for Tonga, PSR, ESAL & EALF, Shear Failure factor for the SAMDM approach, and RTL research work for an overall understanding of pavement behaviour under various load scenarios. There are many researchers who have carried work and are expert at this such as Arnold 2004, Werkmeister 2003, and Araya 2011 that could assist.

At the core of this, which is the original motivation for this research, is the aggregate that is being used. When vehicles travel along pavements, loads are transferred to the pavement via tyres passing on vertical and horizontal stresses and strains, shear stresses and strains. Aggregate reacts to this by way of rotating (rotation or principle stresses apply), reorientation and no rotation where the shear stress reversal occurs.

The key to pavement longevity is how the stresses are absorbed by the pavement layer. Some of the stresses are recoverable, resilient or elastic strains, or non-recoverable, permanent or plastic strain. The magnitude of permanent strain or resilience determines how long the permanent can withstand the designed load.

Flexible pavement designs for UGL have to take into account the prevention of subgrade deformation as well as the durability of the aggregate used. However, design methods traditionally incorporates the tensile strain at the bottom of the surfacing layer, the tensile strain at the bottom of the granular material (top of the subgrade) and the compressive strain on top of the subgrade.

Research work by Araya (2011) highlighted the SAMDM design approach where the structural capacity of the aggregate material in included in the design. Safety Factors (SF) were then determined for all layers in order to counter the design loads. This was considered appropriate for the Tonga situation because aggregate strength can have a direct effect on the SF. Therefore it is possible in future to work on improvement factors for Tonga's limestone to fit in with this approach.

Aggregate behaviour under stress emphasises how critical aggregate shear strength is to resisting permanent deformation. TNZ M/4 grading reflects a well graded mix of aggregate sizes which is in turn reflected on the crushing resistance value it is subjected. The strength and interlocking ability of the pavement aggregate is the key to absorbing load stresses. Thom and Brown 1988 demonstrated that crushed limestone reduces the aggregate stiffness and resistance to permanent deformation.

Werkmeister (2003) stated that grain deformation within aggregates determines resilient deformation. The inter-particle connections transmit forces from load increasing the inter-particle contact area, therefore increasing the resistance. As the contact forces increase ΔF, the displacement $\Delta\delta$ between particles decreases, particle resilient deformation. Critical properties of the aggregate mineralogy, grain size and particle strength determines the magnitude of permanent deformation.

Permanent deformation is caused by a mechanism of re-orientation, characterized by rotation and sliding of the individual particles, resistance to particle sliding and rotation is dependent on the inter particle friction.

When contact stresses between grains exceeds the strength of grains, crushing occurs. This is critical to the Tongan situation. Not just for permanent deformation but also for the aggregate application as chipseal material.

Thom and Brown argued that there isn't a direct link between shear strength because the stress levels between individual grains under loading were different therefore different micromechanical processes must be in play but it needs further work for proof.

Australian standards were specified for material testing in 1994, 1995 and 2010. Some of which were incorrectly assigned such as AS1289 which is for soils not pavement aggregates.

The overall theoretical perspective highlights a number of complex interaction between vehicle loads, the stresses impacted on pavement structures, how the aggregates within the pavement behave in absorbing and dealing with the stresses. Aggregate properties such as grading, shear strength, moisture content, aggregate roughness, mineralogy as well as microscopic level interactions are all part of this process. It is a complex interaction therefore specific tests applied to develop knowledge on how materials used for specific projects would respond is essential. Applicable information specific to the sites concerned such traffic loading, vehicle types and future population growth is crucial.

7.3 Conclusions

7.3.1 Objective 1

All three quarries failed to meet the Source Properties requirements for both TNZ M/10 basecourse and the TNZ M6 chipseal.

The percentage of fines produced were 17.1 %, 18.1 % and 20.9 % which is much higher than the < 10% for TNZ basecourse and chipseal.

As a basecourse material with excessive amount of fines, research reviewed above demonstrated that density, modulus of elasticity and permanent deformation are affected by this. Higher fines attract moisture into the pavement which then reduces density and increase permanent deformation.

The Weathering Quality Index of BB, with 91-95% of aggregate retained on the 4.75mm sieve, these aggregates are not suitable to be used for chipsealing work. Combined with the low Crushing Resistance values, estimated at 30 to 35kN by the OICL technicians, it is not good enough to be used as basecourse material either.

The Particle Size Distribution indicated that at the lower end from 600µm sieves down, the tested material fitted with the TNZ/M10 envelope. From 600µm to 19mm sieves however, the material envelope either sits at the top of the higher limit or exceeds it meaning a high percentage of middle size aggregate. It falls outside the specified envelope on the last sieve size.

The approach and tests carried out for this work was a relatively simple process. However there is enough evidence from this work and in conjunction with tests carried out by others, reviewed above, to draw conclusions and address the objectives set out in Chapter 1.

7.3.2 Objective 2

The contract specifications did not reflect the local conditions, materials and standard based on the test results.

AS1289 was incorrectly specified as the standard test for pavement aggregate. AS 1141 is the correct standard for sampling and testing aggregate. However when tested under AS 1141 the material did not meet the specifications.

Aggregate quarrying creates a number of environmental issues that are of concerns primarily to the local communities concerned and also the country as a whole. Scarring of the landscape, traffic noise, and dust have immediate effects on the community. Quarrying in Tonga invariably breaches groundwater which normally prompts a halt or an end to the operation. There are no restoration requirements therefore the land is completely lost to agriculture or any other use contradicting the government economic drive initiatives.

No future sustainable options were identified during this research. However, exploring and trialling products such fibre, glass, and/or stabilisation of base materials are possibilities for future work.

7.4 Recommendations

Further work will enable the development of better performance aggregate therefore longer lasting pavements enabling the Government of Tonga to focus on economic drive and the resultant improvement in people's lives long term.

1. Acceptance by the appropriate government ministry and relevant staff the fact that Tonga's limestone aggregate properties are inferior to the specified contract requirements.

2. Establish by way of a comprehensive aggregate testing programme trialling combinations of fibre or glass with limestone with a view to introduce appropriate material specifications for Tonga.

The testing programme can be in the form of road trials combined with laboratory experimental work utilising a selected methodology such as the Repeated Load Triaxial (RLT) or the Finite Element Modelling (FEM) developed by Arnold (2003).

This will facilitate a more in-depth understanding of how Tonga's limestone reacts under pressure from the various loading scenarios specific to local conditions and not Australia, New Zealand, Japan, China or any other donor country.

3. Water impacts pavement design life. Tonga is exposed to cyclone and flooding for three to four months a year. Identification of risk areas and construction of effective stormwater systems is essential.

4. Most quarry operations are privately owned and operated. In order for the process and quarry products to be of consistent quality, national standards need to be established. Standardising the extractions and source testing process is recommended.

5. The introduction of regular survey and monitoring of quarry excavation depths, to avoid the risks of breaching watertable, is to be included.

6. National standards approval to operate quarries, compulsory registration and equipment quality assurance approval to form part of the approval process.

7. International consultants and contractors must follow a nationally engagement approval process.

8. Further research into the construction and performance of chipseal and asphaltic concrete surfacing in Tonga. A standardised approach to this decision making process needs to be established. Specifically, research into water in chipseal by NZTA in 2015 could form the basis for further research into the Tonga's situation.

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