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THE EFFECT OF CONCRETE STRENGTH AND FIBRE CONTENT ON THE FATIGUE PERFORMANCE OF ULTRA THIN CONTINUOUSLY REINFORCED CONCRETE PAVEMENT (UTCRCP)

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Dissertation Summary

THE EFFECT OF CONCRETE STRENGTH AND FIBRE CONTENT ON THE FATIGUE PERFORMANCE OF ULTRA THIN CONTINUOUSLY REINFORCED CONCRETE PAVEMENT (UTCRCP)

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Degree	Master of Engineering (Transportation Engineering)

Ultra Thin Continuously Reinforced Concrete Pavement (UTCRCP) is an innovative road paving technology that can have significant advantages over traditional road paving techniques. Full scale testing has shown that UTCRCP can carry in excess of one hundred million E80s (standard 80 kN axle loads). The concrete used for the construction of UTCRCP is high strength and contains steel fibres. The purpose of this study is to investigate the effect of varying the concrete strength and fibre content on the mechanical properties of the concrete used to construct UTCRCP.

In this study testing is restricted to laboratory methods. Fibre reinforced concrete is often tested with the same tests as those used for plain concrete. These are not adequate to fully capture the effects of the steel fibres. A number of test methods are used in this study and their suitability and shortcomings are discussed. Additionally, fatigue tests are conducted and a number of performance measures are used to track the fatigue damage.

It was found that higher concrete strength and high fibre content did not necessarily result in highest performance in all cases. It was also found that there is not a single concrete strength or fibre content that always results in the optimum performance in all cases. For example the concrete strength and fibre content that produced the best results for tensile strength was not the best mix for energy absorption at high deflections. When selecting the concrete strength and fibre content for use in construction the application of the concrete element must be carefully understood. Based on this the designer can then select which of the concrete performance characteristics are of most importance to the desired application. Once this is done an appropriate concrete strength and fibre content can be selected.

Declaration

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I understand what plagiarism is and I am aware of the University's policy in this regard:

The work contained in this thesis is my own original work:

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- My friends and family for their continued support and encouragement.
- “Confitemini Domino quoniam bonus.” (Taizé, 2015)

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Characteristic strength	The compressive strength of concrete that 95% of the concrete will exceed.
Energy absorption	The energy applied to a specimen calculated as the area under the load deflection curve, up to a specific deflection.
First crack strength	The strength at which the load deflection behaviour ceases to be linear
HVS	Heavy Vehicle Simulator.
E80	Equivalent standard axle (80 kN). A unit used to quantify vehicle loads on a pavement structure.
Hysteresis energy	The irrecoverable energy lost between the loading and unloading of a specimen. Calculated as the area enclosed by the loading and unloading curve.
Mean strength	The average compressive strength of concrete.
MOR	Modulus of Rupture.
Principal crack	The point at which the tensile strength capacity of the concrete is exceed. This point is reached subsequent to the first crack strength. If the sample is unreinforced this will represent the ultimate load on the specimen. In a sample with steel reinforcing the load will decrease after this point for a short while before increasing to the ultimate load.
SANRAL	South African National Roads Agency Limited.
Tining	Texturing the surface with a number of parallel grooves. In concrete pavements this is done to improve skid resistance.
UTCRCP	Ultra Thin Continuously Reinforced Concrete Pavement.

1 Introduction

1.1 Background

Transportation is a vital element of any economy. Connecting raw materials to industry and industry to markets creates opportunities. The ability to move goods quickly, safely and efficiently gives business its competitive edge. This can result in more job creation and more opportunities for society. However, increasing traffic volumes and loads result in faster deterioration of the roads condition and thus more frequent road maintenance is required.

Over 70% of the South African road network has exceeded its 20 year design life (Kannemeyer et al., 2007). In order to cater for increasing traffic on an aging road network significant structural strengthening will be required. With limited budget available to perform maintenance of the road network there is increased demand for innovative and cost effective maintenance and rehabilitation options.

It is in this context that Ultra-Thin Continuously Reinforced Concrete Pavement (UTCRCP) has been investigated as a potential option for the structural rehabilitation of roads. In addition to being ideally suited to the South African situation, this technology holds a number of advantages over the current rehabilitation methods.

Scope exists to refine the design of UTCRCP. For example increased fibre content and concrete strength result in concrete that is more expensive. Higher strength concrete is less workable, likewise increased fibre content also reduces workability. Therefore increasing fibre content and concrete strength not only results in a product that is more expensive per unit volume, but the handling and placing difficulties also result in higher construction cost. Workability of the concrete can be increased by the appropriate use of concrete technology however, this also has cost implications. If the fibre content and concrete strength can thus be reduced this will result in a cost saving in the construction cost of UTCRCP. The risk however is that reducing concrete strength and fibre content will compromise the long term performance of UTCRCP. Understanding these interactions will allow for an optimised mix design, reducing cost whilst not compromising on long term performance.

1.2 Objectives of the Study

The main objectives of this study are to:

1. Determine the effect of fibre content and concrete strength on the properties of fibre reinforced concrete.
2. Compare the effectiveness and limitations of various test methods used to determine the mechanical properties of fibre reinforced concrete.
3. Find the concrete compressive strength and fibre content for optimal mechanical behaviour of UTCRCP.

1.3 Scope of the Study

As the aim of the study is to determine the effect of fibre content and concrete strength on the properties of fibre reinforced concrete, the scope of the study is limited as follows:

This study is limited to UTCRCP for high volume roads and while the principles applied or findings may be transferable, the study does not include UTRCP that is applicable to low volume roads (Definitions as per Perrie and Rossman, 2009). This study also does not consider Ultra-thin whitetopping. Ultra-thin whitetopping shares many characteristics with UTCRCP, in that it is thin concrete pavement structure that often contains fibres, however, it is fundamentally a jointed concrete pavement technology (Rasmussen and Rozycki, 2004)

The purpose of this study is not to develop transfer functions[1] for UTCRCP.

Comparisons of performance are based solely on mechanical behaviour. While economic considerations are vital, no cost comparisons or economic comparisons will be made in this study.

Due to cost and time constraints this study is strictly limited to laboratory based testing only. No full scale, or accelerated pavement testing is done.

The only variable to be tested in relation to the steel fibres is the fibre content. Different fibre types, geometry or fibre materials are not be considered in this study.

No testing with regard to different steel mesh spacing or mesh sizes is conducted. In the slab samples where mesh is included to replicate the constructed samples fixed mesh spacing are used.

[1] Transfer functions are used in the South African Mechanistic Pavement Design Method (SAMPDM) in order to predict the number of load applications to failure of pavement layers (Theyse et al., 1996).

1.4 Methodology

A review of the current literature on the topic is undertaken to gain an understanding of the current status of UTCRCP. In addition, fibre reinforced concrete is investigated, along with its performance with respect to fatigue loading.

Secondly, an experimental program is developed. In this program a set number of concrete strengths and fibre contents are selected. A batch of samples are prepared from concrete for each selected fibre content, and each selected concrete strength. A number of tests are then performed on these samples. Tests conducted include; compressive strength, tensile strength, stiffness, flexural strength and fatigue behaviour.

The results obtained are then scrutinised to determine the effect of changing fibre content and concrete compressive strength. In this manner an understanding of the effects of these two variables on the basic mechanical properties is obtained. Where two tests measure the same or similar parameters comparison between of the two methods is made. Finally, the full set of results, with an emphasis on the results of the fatigue based tests, is used to recommend an optimum design for UTCRCP.

1.5 Organisation of the Report

This report is arranged into the following sections:

- Chapter 1 serves as the introduction to the report.
- Chapter 2 contains a review of literature available on UTCRCP, fibre reinforced concrete and fatigue in concrete.
- Chapter 3 contains the methodology followed in the experiments conducted for this research. This includes the experimental design as well as the details of each test used to evaluate the performance of the concrete
- Chapter 4 contains the results obtained from the tests conducted, as well as a discussion of the results
- Chapter 5 is the concluding chapter where the findings are summarised.

2 Literature Review

2.1 Introduction

UTCRCP is a road building technology that is being investigated in South Africa. The results of these investigations have demonstrated the potential of this technology. This section of the report summarizes the research on UTCRCP to date. This includes research on the material behaviour of the various components, as well as the methods used to test fibre reinforced concrete. These tests have shown some areas of concern where further research should be done before wide scale implementation is undertaken.

2.2 Background

Perrie and Rossman (2009) define UTCRCP as a thin concrete layer, with a thickness in the region of 60 mm constructed from concrete in the order of 100 MPa that contains both steel and plastic fibres. The concept of UTCRCP was modified from a technique used to construct industrial floors. The method was successfully used to rehabilitate a bridge deck in the Netherlands. This demonstrated its potential as a road pavement. Apart from research conducted in South Africa, some research into UTCRCP has also been conducted in France where some accelerated pavement tests have been done. A trial section on the road to a quarry in France has also been constructed.

In South Africa the capabilities of the UTCRCP and its performance were investigated using the Heavy Vehicle Simulator (HVS). Trial sections were constructed near the mass control centre (weighbridge) at Heidelberg[2]. One of the screener lanes of the weighbridge was constructed from UTCRCP. After the conclusion of the first round of HVS tests some laboratory experiments were conducted in conjunction with some finite element modelling. Following this, a second round of HVS tests were conducted at Heidelberg (Kannemeyer, et al., 2007).

Trial sections of UTCRCP have been constructed west of the Huguenot tunnel near Paarl on the N1 and on the N12 in Johannesburg. These tests all contributed to the development of UTCRCP under South African conditions and using South African materials.

Perrie and Rossman (2009) define two types of Ultra-thin Concrete pavements; firstly, UTCRCP for highly trafficked pavements: and secondly, Ultra Thin Reinforced Concrete Pavement (UTRCP) for lower trafficked pavements. While the two types of thin concrete pavements share many features, they

[2] The experiments were conducted at Heidelberg in the Gauteng Province of the Republic of South Africa

have some distinct differences. The research conducted as part of this project focused on UTCRCP, the type intended for highly trafficked pavements.

Many components go into the construction of UTCRCP. These include concrete, steel fibres and steel mesh. In the next sections each of these materials is discussed. Then the issues regarding the construction of UTCRCP will be discussed, and finally, issues regarding fatigue testing and fatigue behaviour of concrete.

2.3 Components

2.3.1 Concrete

UTCRCP, being a concrete paving technology, is chiefly composed of concrete. Due to the limited depth of the layer, the high performance requirement, the steel fibres and the reinforcing mesh, the specifications of the concrete are stringent.

The required compressive strength of the concrete used for the construction of UTCRCP is between 90 and 120 MPa. This means that the concrete constituents need to be carefully sourced and batched. In order to meet the high compressive strength requirement a low water cement ratio is used. During the construction of the UTCRCP experimental sections on screener lanes at Heidelberg, a water cement ratio of 0.325 was used. This water cement ratio combined with superplasticisers and a suitable mix design, produced concrete that achieved both sufficient strength and workability (Mukandila et al., 2009).

Laboratory tests have shown that the optimum slab thickness is between 50 mm to 60 mm in order to maximise bending resistance (Kannemeyer, et al., 2007). Reinforcing mesh needs to be placed at the centre of the UTCRCP. Placing concrete within these tight confines limits the maximum aggregate size that can be used in the concrete. The experimental sections constructed at Heidelberg used 6.75 mm stone (Kannemeyer, et al., 2007; Mukandila et al., 2009).

The binder is a critical component of any concrete. The first experimental sections of UTCRCP constructed at Heidelberg used an imported cement binder. This binder is used in the construction of industrial floors. Following the results of the first round of HVS testing, locally available cement was used in the construction of the second set of experimental sections. This was blended with both fly ash (FA) and Condensed Silica Fume (CSF). This binder matched the performance of the imported binder (Kannemeyer, et al., 2007). It was therefore decided that locally produced cements are suitable for use in UTCRCP.

During curing, concrete shrinks, because this shrinkage is restrained the reduction in volume places the road pavement in tension. De Larrad (2005) observes that the proper control of concrete shrinkage can be used to the advantage of engineers. Placing the pavement in tension assists in resisting the buckling of the pavement that results from excessive thermal expansion.

2.3.2 Mesh

In UTCRCP reinforcing mesh is placed on the centre of the concrete layer. The bars are both in the longitudinal and transverse direction of the pavement. The pavement is continuous, thus there are no expansion joints as in a conventional jointed concrete pavement. The concrete will expand and contract due to environmental conditions such as temperature and moisture. The purpose of reinforcing mesh in a concrete pavement is not to prevent cracking but rather to control the cracking that will occur. The width and distribution of the cracks should be controlled by the steel, in order to preserve the load carrying capacity of the slab and prevent ingress of water (Perrie and Rossman 2009).

As part of the Heidelberg tests, experiments were done in order to determine the optimum steel diameter. It was found that increased steel diameter resulted in increased bending resistance. However, thicker mesh also resulted in higher cost and increased the difficulty of placing the concrete. The best mesh diameter recommended by Kannemeyer et al., (2007) was 5.6 mm (Y6) as a good compromise between cost and constructability.

Mesh spacing is a further variable that can be altered. A number of different mesh spacings have been used. The Screener lanes at Heidelberg used 100 mm by 50 mm spacing mesh (Mukandila et al., 2009). Kearsley and Mostert (2012) compared mesh spacing of 50 mm x50 mm, 75 mm x 75 mm and 100 mm x 100 mm. Following the results of the HVS testing, a mesh spacing of 50 mm by 50 mm was recommended as the best balance between cost constructability and risk (Kannemeyer, et al., 2007). A further finding was that placing longitudinal steel closer to the top of the layer reduces compressive stress at a joint or a crack, reducing the risk of spalling (Kannemeyer et al., 2007).

An experimental section of UTCRCP was constructed on the N12 in Johannesburg. High shrinkage from the concrete was experienced during curing, placing high tensile forces on the reinforcing mesh. These forces were high enough to snap the reinforcing mesh (Perrie et al, 2011). The mesh used at this particular section was made from 3 mm diameter high tensile cold drawn wire steel. The mesh was mechanically sufficient to act as reinforcing for the bending moments resulting from traffic loading. Cold drawn steel has high yield strength but it does not possess much in the way of ductility. Laboratory testing showed that this mesh did not increase the post crack load caring capacity of the UTCRCP (Kannemeyer et al., 2007) This occurrence not only highlights the magnitude of the forces in the UTCRCP but also the importance of supplying sufficiently ductile mesh reinforcing.

2.3.3 Fibers

The loads applied on concrete pavements result in bending stresses on the concrete. In order to resist bending, a material must have both compressive and tensile strength. Concrete is a material that is strong in compression; however, in tension it is relatively weak. When steel fibres are mixed into concrete the tensile performance of the concrete can be improved.

When the ultimate tensile stress of unreinforced concrete is exceeded, it will fail in a brittle manner. In this cracked condition, the concrete will no longer be able to carry loads in bending. The inclusion of steel fibres can delay and control the tensile cracking in the concrete. This vastly increases the load carrying capacity of the concrete (Elsaigh et al., 2005; Chen, 2004).

The first UTCRCP experimental sections at Heidelberg used 12 mm straight fibres. On these sections it was seen that once cracking had developed failure occurred rapidly. These sudden failures were believed to be as a result of the fibres being of the incorrect type and therefore were unable to effectively transfer load across the cracks. In the second set of trial sections constructed, the original 12 mm straight fibres were replaced with a 30 mm hooked end fibres. These fibres were found to be more successful in preventing the sudden failures (Kannemeyer et al., 2007). In laboratory experiments carried out as part of the HVS tests, the 30 mm fibres increased the bending stress by 25% (Kannemeyer et al., 2007).

The function of the Polypropylene fibres added to the concrete is different to that of the steel fibres. Polypropylene fibres provide very little contribution towards the flexural performance of concrete (Zhang, 1998). The purpose of the polypropylene fibres is to modify the properties of the fresh concrete. In particular, these fibres have been found to reduce plastic shrinkage cracking of the concrete (Illstone and Domone, 2008).

A more detailed discussion of further effects of fibres and their performance is presented under subsequent headings in this chapter.

2.3.4 Construction

Constructing a UTCRCP presents some challenges. Tolerances are very tight when the depth of the pavement layer is 50 mm. The dense steel mesh makes placing and compaction of the concrete difficult. In addition, the high strength concrete requires very low water cement ratios which, in turn, reduces the workability of the concrete. During construction of the Screener lanes at Heidelberg the slump of the concrete was between 90 and 208 mm before the addition of fibres (Mukandila et al., 2009). Workability is further reduced by the presence of the steel fibres. Thus the construction of UTCRCP is more demanding than that of ordinary reinforced concrete.

During construction of the HVS test sections at Heidelberg it was found that a ready mix truck could not supply enough energy to properly mix the concrete. A drum mixer was then used with great success (Kannemeyer et al., 2007). Construction of subsequent sections was conducted using a pan mixer to good effect (Mukandila et al., 2009). When constructing UTCRCP screener lanes at Heidelberg Mukandila et al. (2009) reported that a 720 kg pan mixer had a production rate of up to 15m³ per day, which was the equivalent of approximately 60m of road length.

During the construction of the various UTCRCP it was found that the most effective method was to place the concrete using a chute and then spread it by hand. Initial compaction done by poker vibrator should then be followed by a double vibrating screed compactor (Kannemeyer et al., 2007) or an oscillating triple roller screed beam (Mukandila et al., 2009). Placing the concrete using an asphalt paver did not produce satisfactory results (Kannemeyer et al., 2007). Laboratory samples of UTCRCP were cut to inspect whether the steel fibres remained trapped on the mesh. It was found that the fibre distribution above and below the bars was similar. This confirms that the fibres are able to flow through the reinforcing mesh (Kearsley and Mostert, 2012). The surface could be finished either by tining or using a stiff broom. Mukandila et al. (2009) reports that stiff brooming gave a more satisfactory finish than tining.

Curing with plastic sheeting was not successful as securing these sheets in windy conditions was found to be difficult. Thus the use of a curing compound is recommended. It was found that a solvent type resin based gave best results. In addition, white pigment was found to give better results than a clear curing compound (Mukandila et al., 2009). This was largely due to the reflectivity of the white pigment lowering the temperature and the ability to visually inspect the consistency of its application.

2.3.5 Deterioration

Before constructing UTCRCP on public roads, designers wanted to have a better idea of the behaviour of this pavement. In order to achieve this, a series of accelerated pavement tests were commissioned. The benefit of these tests was that the anticipated useful life of the pavement under traffic loads could be estimated. Different configurations could be tested in order to compare their performance and determine the optimum design.

Trial sections of UTCRCP were constructed. These pavements were then instrumented with a number of different sensors and loading was applied using the HVS. This equipment contains a wheel load which travels back and forward across the test section (Kannemeyer et al., 2007).

From the observations of these experiments, in conjunction with finite element modelling, the mechanism of deterioration of the UTCRCP was identified. In the pavement, transverse cracks will form as a result of shrinkage of the concrete paste during curing or at construction joints. When the wheel load is directly above one of these cracks high tensile stresses develop at the bottom of the pavement. This stress needs to be transferred by the action of the steel fibres bridging the crack. At the same time compressive stresses are a maximum at the top of the pavement that can result in spalling of the concrete. With the wheel above the crack a second high tensile stress zone develops at the top of the pavement near the edge of the tyre contact patch. The FEM modelling also showed that when the wheel is 450 mm from the crack, high tensile stresses develop in the base of the UTCRCP directly below the tyre. This leads to bottom up cracking of the pavement at this point. However, the HVS results indicated that this cracking was not the main source of deterioration of the pavement (Kannemeyer et al., 2007).

Research by Wang et al. (1997) found that cracks of less than 50 microns have little effect on concrete permeability but once cracks exceed this threshold permeability increases rapidly. Once the cracks have opened to an extent where water is able to penetrate the pavement, pumping of the layer below can occur. This leads to the formation of a void between the UTCRCP and the supporting layer. The void, in turn, leads to higher deflections and consequently to higher stress in the UTCRCP. This cycle, if allowed to continue, leads to failure of the pavement (Kannemeyer et al., 2007).

When tested dry the UTCRCP carried 92 million E80s without showing any significant sign of distress. Distress only began to occur when water was added to pavement sections to simulate wet climatic conditions. The researchers concluded that had it not been for addition of water that led to a weakening of the support, the pavement would not have failed (Kannemeyer et al., 2007). When tested under wet conditions the pavement was able to carry 73 million E80s (Perrie et al., 2011). Where grouting was applied as a repair method to fill voids below the pavement, the section carried 104 million E80s (Perrie et al., 2011). This demonstrates the longevity of UTCRCP.

Curling of concrete pavements can be a concern, but HVS testing found that curling was not a significant factor in the deterioration of UTCRCP (Kannemeyer et al., 2007). However, another temperature related deterioration was observed on the UTCRCP screener lanes at Heidelberg. A curved portion of the lane detached from the foundations. This was clearly a temperature related affect as in hot weather the pavement was observed to lift clear of the foundations. To this point this phenomenon had not been observed on any other portion of UTCRCP (Mukandila et al., 2009). It was then seen more dramatically on the UTCRCP trial section on the N1 near the Huguenot Tunnel. High temperatures, coupled with moisture, were concluded to cause expansion of the concrete to such an extent that the UTCRCP buckled under the compressive force (Van Zyl, 2011).

2.4 Effect of Fibre Content and Type on Fibre Reinforced Concrete Properties

Plain Concrete is a brittle material. The addition of steel fibres can give concrete ductility. The greater the quantity of fibre added to the concrete, the greater the modification of the mechanical behaviour. Likewise, changing the geometry and dimensions of the fibre will also affect the extent to which the mechanical properties of the hardened concrete are altered. Fibres can be manufactured from many different materials including: steel, polypropylene, glass and natural fibres. Only steel fibres are discussed in this review.

Concrete is strong in compression. It is generally accepted that the addition of steel fibres to concrete provides insignficant increase in compressive strength (Illstone and Domone, 2008). However Song and Hwang (2004) found that increasing the steel fibre content did have some effect on the compressive strength of concrete. They found that by increasing the steel fibre content from 0.5% by volume to 1.5% by volume the compressive strength of an 85 MPa plain concrete mix could be increased to 98 MPa. When the steel fibre content was further increased to 2% by volume a slight decrease in the compressive strength was recorded (see Figure 1). Kearsley and Mostert (2010) also measured this decrease in compressive strength at fibre contents of 1.5% by volume and higher. They attributed this to the difficulty in fully compacting the concrete with the high concentration of fibres. The presence of fibres can increase the pore and initial microcrack density; and this can also lead to decreased compressive strength (Grzybowski and Meyer, 1993).

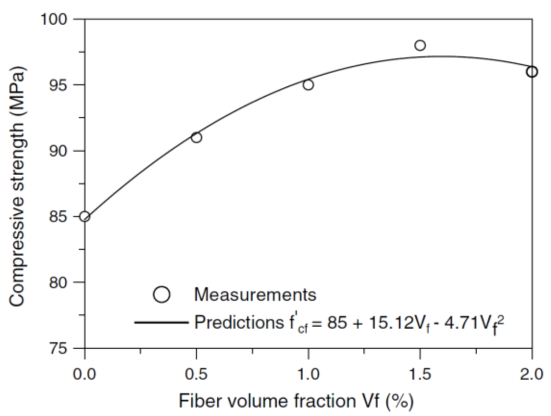

Figure 1: Effect of Fibre Volume on Compressive Strength (Song and Hwang, 2004)

Relative to its compressive strength, concrete is weak in tension. In general the tensile strength of concrete is between 8% and 11% of the compressive strength (Illstone and Domone, 2008). When direct tensile strength of concrete is measured, the addition of steel fibres is reported to have little effect (Illstone and Domone, 2008). However, when tensile strength is measured by indirect tensile tests such as the splitting tensile strength, measured on cylinder specimens, or the Modulus of Rupture (MOR) test measured on beam specimens, then fibre content plays an important role (Illstone and Domone, 2008). Song and Hwang (2004) measured an increase in tensile splitting strength and MOR with similar results (Figure 2 and Figure 3). Johnston and Zemp (1991) found that the first crack and ultimate strengths of beams in flexure were significantly improved by the addition of steel fibres of 0.5% and 1.5% by volume. The purpose of adding steel fibres to concrete is thus to delay and control tensile cracking of the concrete. The result of adding steel fibres is that tensile forces are able to be transferred after the concrete has cracked. It is assumed that the fibres will be distributed randomly through the concrete mix. Increasing the quantity of fibres added to the mix will increase the probability of having a single or multiple fibres bridging the crack that could form as a result of tensile stresses. Thus, increased fibre content improves tensile performance. Adding sufficient fibre to concrete will give the cracked concrete a greater load carrying capacity than the tensile cracking strength (Elsaigh and Kearsley, 2002).

Under wheel loads, concrete road pavements experience bending. Bending results in both compression and tensile forces being induced in different parts of the concrete, and it is the weaker tension zones where the fibres provide the most benefit. In laboratory experiments carried out as part of the HVS tests the 30 mm fibres increased the bending stress by 25% (Kannemeyer et al., 2007). During the Roodekrans experiments it was found that 30 kg/m³ of steel fibres was sufficient to construct a concrete pavement that could out-perform a plain concrete pavement even whilst being 25% thinner (Elsaigh et al., 2005).

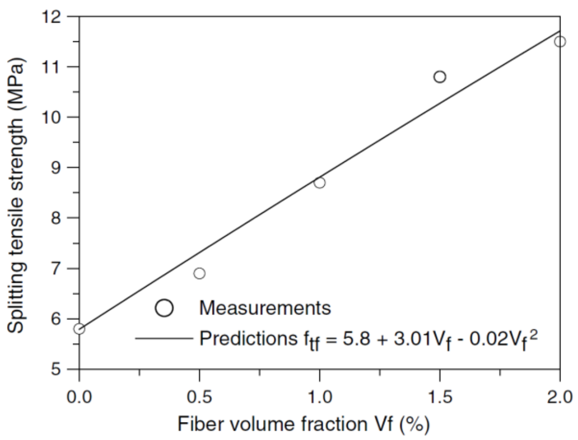

Figure 2: Effect of Fibre Volume on Splitting Tensile Strength (Song and Hwang, 2004)

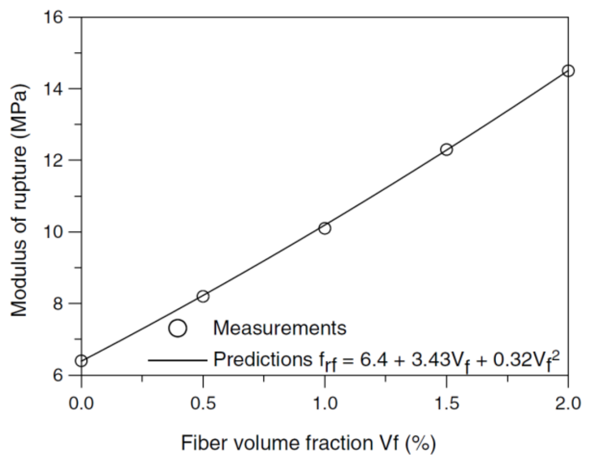

Figure 3: Effect of Fibre Volume on Modulus of Rupture (Song and Hwang, 2004)

In flexure the concrete pavement experiences both tensile and compressive forces. In order to transfer load the material must offer both tensile and compressive strength. Since concrete performs well in compression, it is the tensile strength of the fibre reinforced concrete that will govern the overall performance of pavements. The full advantages of using fibre reinforced concrete for flexural reinforcement is only gained in statically indeterminate structures where the formation of plastic hinges allows for the redistribution of stresses (Nemegeer, 1996). Slabs placed on the ground such as road pavements are able to take full advantage of this. After initial cracking has occurred, the action of the fibre allows for the redistribution of stress. This allows the slab to act in a ductile manner, the greater the post crack load carrying potential of the material the greater the ductility of the slab. In this condition the concrete is beyond the linear elastic range and for this reason it is not appropriate to use linear elastic theory to predict the behaviour of fibre reinforced concrete (Elsaigh et al., 2005).

Yoo et al. (2015) reports a clear increase of flexural strength with increased fibre content. Increasing fibre content not only increases maximum strength but also ductility (Zhang and Stang, 1998).

Fibre content is not the only variable that can be changed. Other factors, like aspect ratio, also influence the strength of the concrete. Aspect ratio is the ratio of the length of the fibre over the width of the fibre. Yap et al. (2015) showed that the flexural strength of concrete increased when aspect ratio was increased. Under monotonic loading fibre content had most effect on first crack strength. Fibre aspect ratio and fibre type also influenced the flexural strength but to a lesser extent (Johnston and Zemp, 1991). Although increasing fibre aspect ratio from 50 to 75 had a positive effect, little additional benefit was seen by increasing the aspect ratio from 75 to 100. Stiffer high-strength wire fibres performed better than ductile low strength slit sheet fibres at aspect ratios over 70. At aspect ratios around 50 similar performances were observed between the two types of fibres (Johnston and Zemp, 1991).

In a study by Abu-Lebdeh et al. (2011) the performance of hooked end fibres and straight fibres was compared. The fibres were the same length, diameter and made of the same material. The hooked fibres not only measured higher peak pull-out loads than the straight fibres, but the hooked fibres also measured higher pull out energy than the straight fibres. This confirms the results of Zhang and Stang (1998) who reported that hooked fibres produce more ductility in flexure than straight fibres. In the study by Abu-Lebdeh et al. (2011), flat end fibres measured higher peak pull-out loads than their smooth counterparts. However, the pull-out energy was higher for the smoothed fibres than for the ones with the flattened ends. This was attributed to the fact that these fibres did not pull-out but ruptured. Helical twisted fibres, having a slightly larger cross-section than the hooked end fibres, measured higher peak loads than the hooked end fibres but lower pull out energy. The lower pull out energy was attributed to the rupture of the helical fibres.

Smooth and deformed wire fibres were found to be more effective in improving flexural performance than melt extract or slit sheet fibres (Johnston and Zemp, 1991).

2.5 Effect of Concrete Strength on Fibre Reinforced Concrete Properties

Steel fibres are embedded in the concrete. It is logical to assume that change in concrete strength will affect the bond between the concrete and the fibres. Abu-Lebdeh et al. (2011) found that increasing the concrete matrix strength increased the performance of steel fibres. The increase was seen both in terms of peak pull out load and pull out energy. This only holds true, however, when the fibres pull out completely and do not rupture. When concrete strength is increased, pull out energy increases by a greater percentage than peak load.

Changing fibre geometry or type can have a much larger influence on peak load and pull out energy than changing concrete strength. Substituting a straight fibre for a deformed fibre had a larger effect on both peak load and on pull out energy. This is particularly true when dealing with hooked end fibres (Abu-Lebdeh et al., 2011).

The interactions with flat and helical fibres are more complex. At increased concrete strengths both flattened end and helical fibres showed an increase in peak load. However, a decrease in pull out energy was recorded. Unlike the hooked end fibres, these fibres ruptured before being pulled out of the concrete matrix. The decrease in pull out energy was attributed to the manner in which these fibres ruptured (Abu-Lebdeh et al., 2011).

Cement extenders are often added to concrete and have many beneficial properties. The cement used for the UTCRCP test sections contained both silica fume and fly ash as cement extenders

(Kannemeyer et al., 2007). Chan and Chu (2004) reported that including silica fume greatly enhanced the fibre bond with the concrete matrix. Their study showed that to maximise the fibre pull out energy the optimum silica fume cement ratio is in the range of 20% to 30%. The bond strength of the fibres to the concrete matrix was found to increase by 11.6% and 13.8% for 20% and 30% of silica fume respectively; the pull-out energy was found to increase by 95.6% and 98.9%. By examining scanning electron photographs of the fibres Chan and Chu (2004) concluded that the increase in pull-out energy was due to material that had adhered to the fibres. This increased the friction between the fibre and the concrete matrix, consequently increasing the amount of energy required to move the fibre through the concrete matrix (see Figure 4).

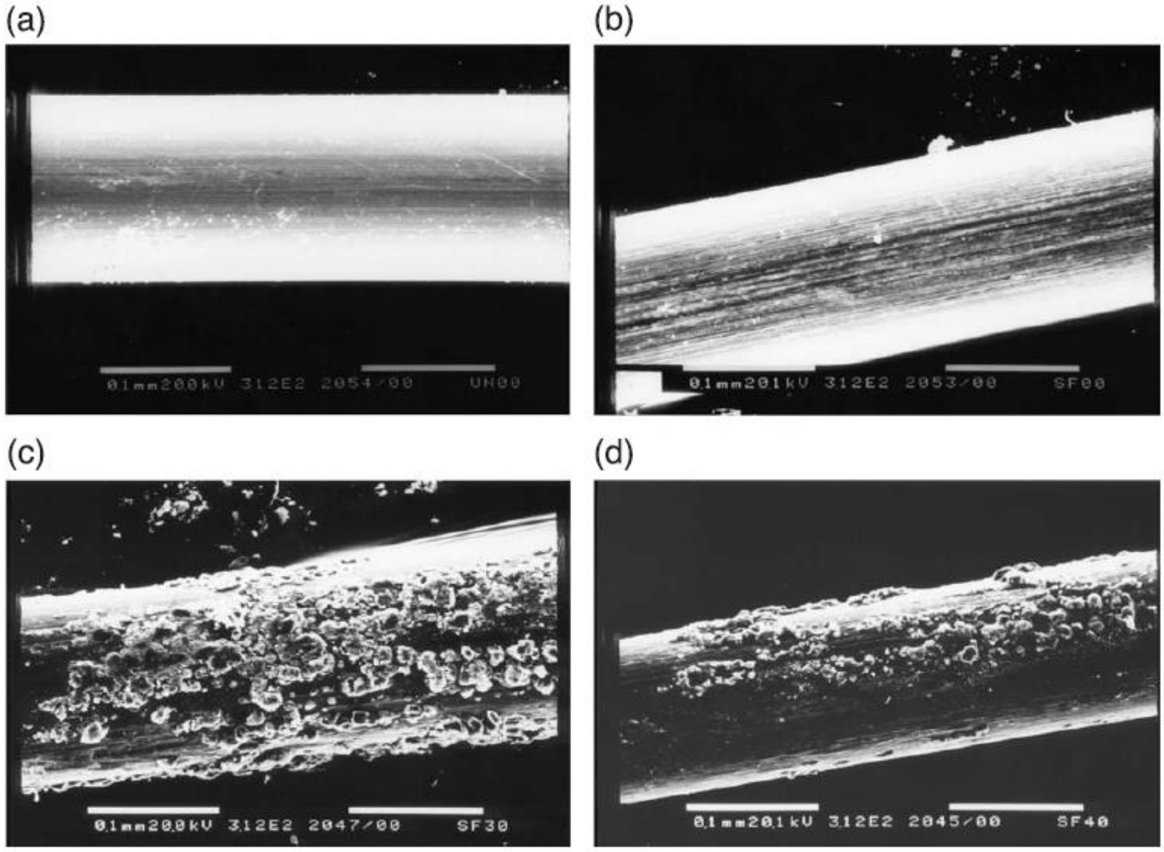

Figure 4: Scanning Electron Microscope Photographs of Steel Fibres at x300 Magnification: (A) Raw Fibre Surface (B) Pullout Fibre Surface (0% Silica Fume) (C) Pullout Fibre Surface (30% Silica Fume) (D) Pullout Fibre Surface (40% Silica Fume) (Chan and Chu 2004)

2.6 Effect of Fibres on Fatigue Behaviour of Concrete

Lee and Barr (2004) define fatigue in concrete as "*a process of progressive permanent internal structural changes in a material subjected to repeated loading. These changes are mainly associated with the progressive growth of internal microcracks, which result in a significant increase of irrecoverable strain.*" Fatigue loading can be placed into two categories (Rilem, 1984):

1. low cyclic loading characterised by few load cycles at high stress levels or
2. high cyclic loading characterised by many load cycles at low stress levels.

The Rilem (1984) report does not further define high or low stress levels or high or low load cycles. A better definition of these load cycle ranges was presented by Hsu (1981) as indicated in Table 1. These range from low cycle fatigue to super high cycle fatigue.

Table 1: Fatigue Cycle ranges (Hsu, 1981)

Low-cycle Fatigue			High-cycle Fatigue				Super-high-cycle fatigue		
1	10^1	10^2	10^3	10^4	10^5	10^6	10^7	10^8	10^9
Structures subjected to earthquakes			Airport pavements and bridges		Highway and railway bridges, highway pavements		Mass rapid transit structures		Sea structures

Fatigue strength is often measured as the fraction of the static strength that can be supported repeatedly for a given number of cycles. These results are commonly displayed in a chart called a stress fatigue life (S-N) curve or a Wöhler curve. Here the maximum stress level applied is plotted against the number of load repetitions until failure. S-N curves are usually plotted for a given constant minimum stress level and probability of failure (see Figure 5). S-N curves can also be plotted for a constant ratio between the minimum and maximum stress levels. This ratio between minimum and maximum stress level is given the symbol R (see Figure 6). These curves often approximate well to a straight line. Stress is often given as a ratio of the ultimate stress of a sample tested under static conditions. (monotonically increasing load). In these tests the rate of loading is often vastly different from the rate of loading applied during the fatigue test. Interpreting the results and using them in design must thus be done with the necessary caution (Rilem, 1984).

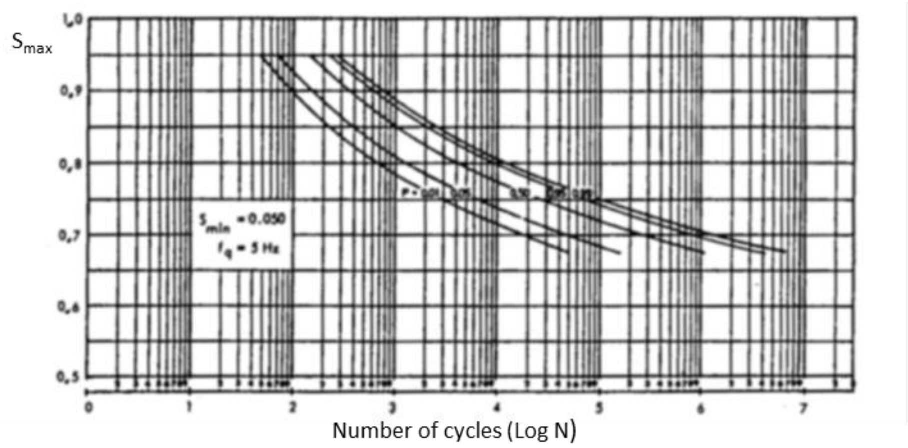

Figure 5: S-N Curve (Rilem, 1984)

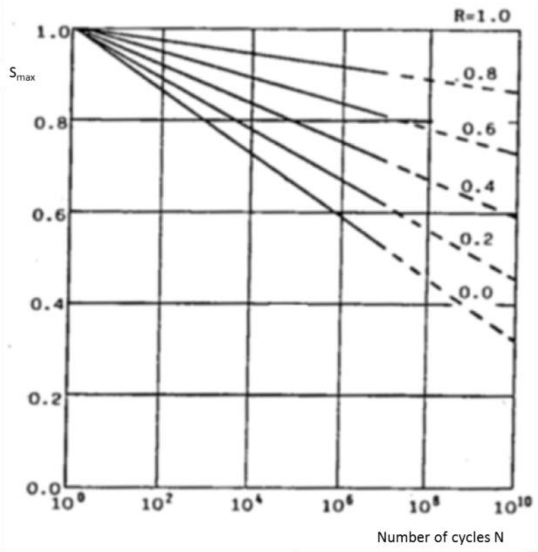

Figure 6: S-N Curves for Constant R-values Where R=Smin/Smax (Rilem, 1984)

Steel and other metals exhibit a fatigue limit; this means that below a certain stress level the fatigue life will be infinite. In plain concrete, however, no fatigue limit has yet been found (Rilem, 1984; Johnson and Zemp, 1991). Cyclic loading can be conducted at a loading frequency between 0.8 Hz and 15 Hz with little effect on the fatigue strength. This range can be used, provided that the stress is lower than 75% of the static strength (Rilem, 1984). Testing at a high frequency is undesirable as temperature build up can occur. It has been reported that fatigue of reinforced concrete loaded at a frequency of 10 Hz was found to lead to a rise in temperature of 8°C (Rilem, 1984). Testing a concrete sample under monotonically increasing load, after it has been subjected to a cyclic load, has been found to be an unreliable measure of fatigue damage. Some samples measured in this manner have been found to have a higher strength than the sample to which no cyclic load was applied. Increases of up to 15% have been reported. Explanations for this phenomenon include greater specimen maturity induced by temperature rise, release of residual stress under the repeated loads, strain hardening, or increase in strength as a result of loss of capillary moisture during the course of loading (Rilem, 1984).

Endurance limit is defined as the maximum load at which 2 million load cycles can be sustained. The addition of steel fibres can increase the endurance limit by as much as 90% for certain types and amounts of fibre. It has also been found, however, that some combinations of fibre types and amounts provide little improvement over plain concrete (Johnston and Zemp, 1991).

Fatigue failure in concrete is considered to take place in three phases:

1. Flaw initiation in weak regions of the concrete matrix.
2. Microcracking - where these flaws increase in size.
3. Fatigue crack growth - where these cracks join together forming continuous cracks (Gao and Hsu, 1998).

Fatigue crack growth in concrete can be categorised into two phases. The first phase is the deceleration phase where the rate of crack growth decreases with the increasing crack length. This is followed by the acceleration stage where the crack growth rate increases steadily up to the point of failure. The decelerating phase has been successfully modelled by the R-curve concept. The accelerating phase was found to be adequately described by the Paris law; the point of inflection where the acceleration phase initiates was found to closely correlate with the crack length of the post peak load of the sample tested under a monotonically increasing load (Kolluru et al., 2000).

The inclusion of steel fibres increases fatigue resistance as they are able to bridge microcracks, and by transferring load across the crack are able to slow the growth of these cracks. This prolongs the fatigue life of the concrete in the second phase of fatigue damage. In the third phase crack growth is slowed as fibre bridging and fibre pull-out dissipates energy in the wake of the crack tip (Lee and Barr, 2004).

The inclusion of steel fibres improves flexural fatigue performance over plain concrete (Zhang and Stang, 1998). When fibre content was increased in the range from 0% to 1.5% by volume, better flexural fatigue performance was achieved as indicated in Figure 7 (Johnston and Zemp, 1991). The presence of large quantities of fibres in the concrete matrix could increase the initial microcrack density. Should this occur the effect could lead to a strength decrease (Grzybowski and Meyer, 1993). Therefore increasing the fibre content of concrete will not necessarily always lead to increased fatigue performance particularly at high fibre volumes. This effect has been encountered experimentally by Zhang and Stang (1998). They recorded that when fibre content was increased from 1% to 2% the flexural performance decreased at load cycles in excess of 100 000 (see Figure 8). This effect is also seen in the fatigue of plain concrete in compression where the addition of fibres marginally decreases its performance (see Figure 9). However, the inclusion of fibre has been reported to significantly improve the bending fatigue of concrete members as indicated in Figure 10 (Lee and Barr, 2004).

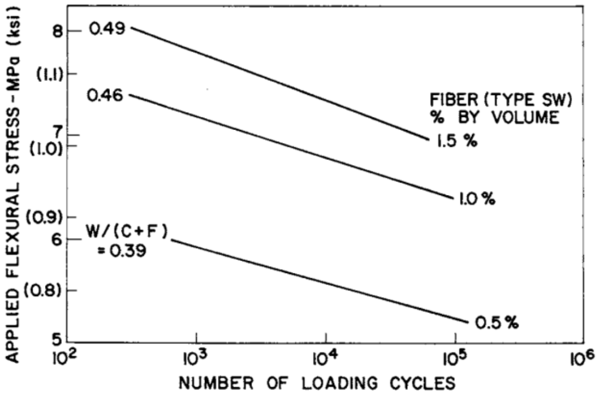

Figure 7: Effect of Fibre Content on S-N Relationships in Terms of Actual Flexural Stress for Smooth Cold-Drawn Wire Fibres (Johnston and Zemp, 1991)

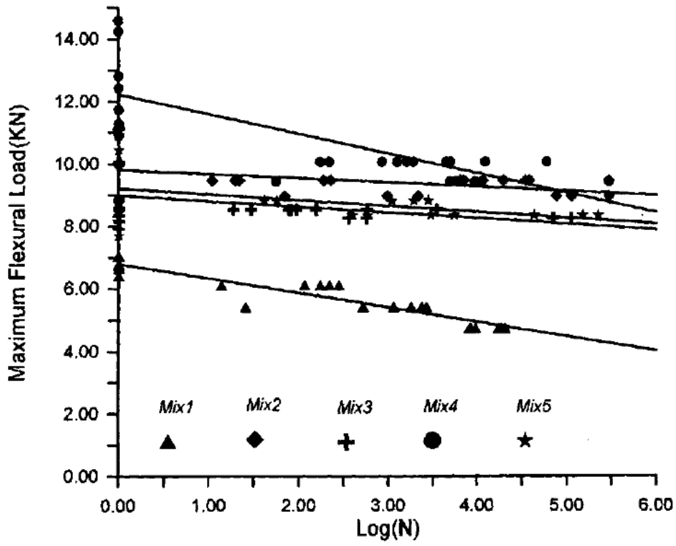

Figure 8: P-N Relationships; Mix 1- 0% Fibre, Mix 2- 1% Smooth Steel Fibre, Mix 3- 1% Hooked Steel Fibre, Mix 4- 2% Hooked Steel Fibre, Mix 5- 1% Hooked Steel Fibre + 1% Polypropolyene Fibre (Zhang and Stang, 1998)

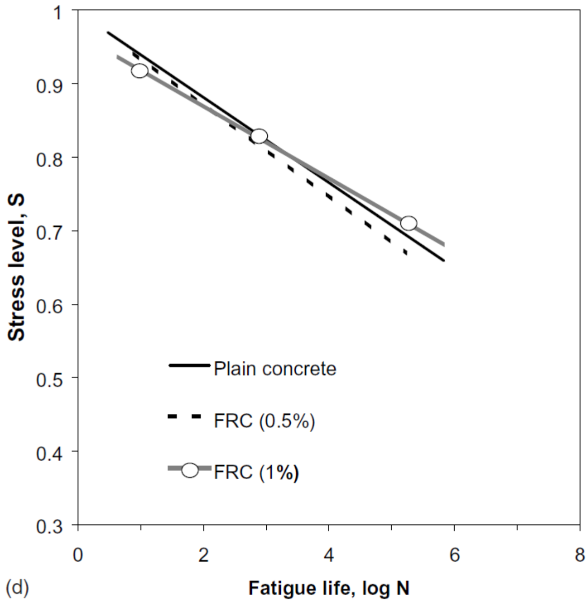

Figure 9: Comparison between Plain and Steel Fibre Reinforced Concrete Under Compression Loading (Lee and Barr, 2004)

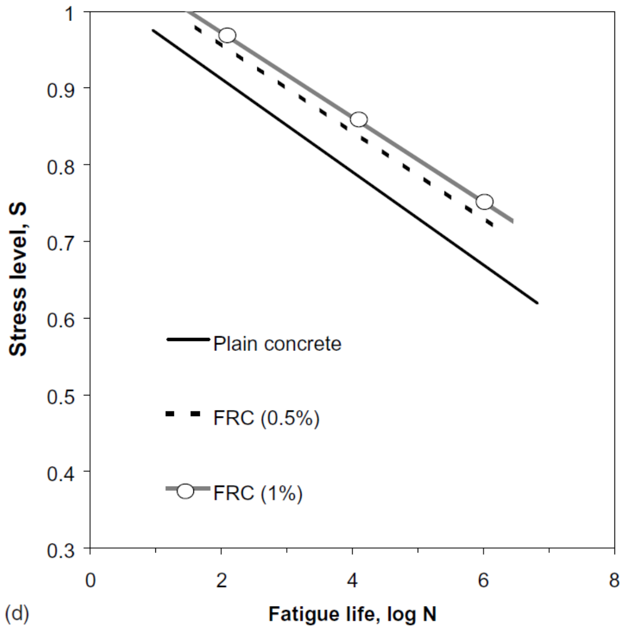

Figure 10: Comparison between Plain and Steel Fibre Reinforced Concrete Under Flexural Loading (Lee and Barr, 2004)

In tests conducted with smooth cold drawn wire fibres increasing aspect ratio gave better flexural performance. However, when fibre aspect ratio was increased from 75 to 100, a decrease in performance was noted. Better fatigue performance was obtained from stiffer high strength wire fibres than ductile low strength fibres. It was also observed that the enhanced performance of the stiffer high strength wire fibres was at aspect ratios over 70. At aspect ratios around 50, high strength and ductile fibres had similar performance (Johnston and Zemp, 1991).

When compared on the basis of applied stress, fibre content is the governing factor of fatigue performance. When compared as a percentage of first crack strength or ultimate strength the effect of fibre content is less obvious (Johnston and Zemp, 1991).

2.7 Hysteresis Behaviour of Concrete

When cyclic loading is applied to a specimen the load deflection behaviour for loading and unloading can be quite different. This behaviour is known as hysteresis. In certain materials the hysteresis behaviour will change over time as more and more load cycles are applied. This is known as memory dependence and can be likened to a counting mechanism that records the cumulative loading applied to the material. Empirical relationships can be established linking the width of the hysteresis loop to the number of loading cycles that will produce fatigue failure (Erber et al., 1993).

In materials that have a memory hysteresis behaviour there are two main modes of hysteresis evolution. These can be thought of as the bounding cases. The first is where successive loops become consecutively smaller, eventually disappearing all together, this is known as fading hysteresis and in this case the energy dissipation is finite. The second is known as drifting hysteresis. This is where loops asymptotically enclose a finite area. In theory the energy dissipation is thus infinite under continued load applications (Erber et al., 1993). In a study by Do et al. (1993) the hysteresis of high strength plain concrete was measured (see Figure 11). In this study the area of the hysteresis loop was used as a measure of the energy dissipated, this measure will closely approximate hysteresis loop width referred to by Erber et al. (1993) (Do,et al., 1993). In Figure 11 it can be seen that the initial response after commencement of the cyclic load is a decrease of energy dissipation. Energy dissipation then stabilises and increases to levels larger than that of the first cycle up to a point of failure. Energy dissipation is calculated as the area enclosed by the hysteresis loop and therefore a similar trend in the data can be expected where loop width is measured. Initially the loop width will decrease and then remain constant. As loading continues the loop width will increase to a level larger than the initial loop width up to the point of failure.

Based on the literature reported it is therefore clear that the evolution of hysteresis behaviour under cyclic loading is a useful measure of the fatigue performance of concrete under cyclic loading. Not only does it give a measure of material damage but could also be used to track and predict the anticipated remaining sample life.

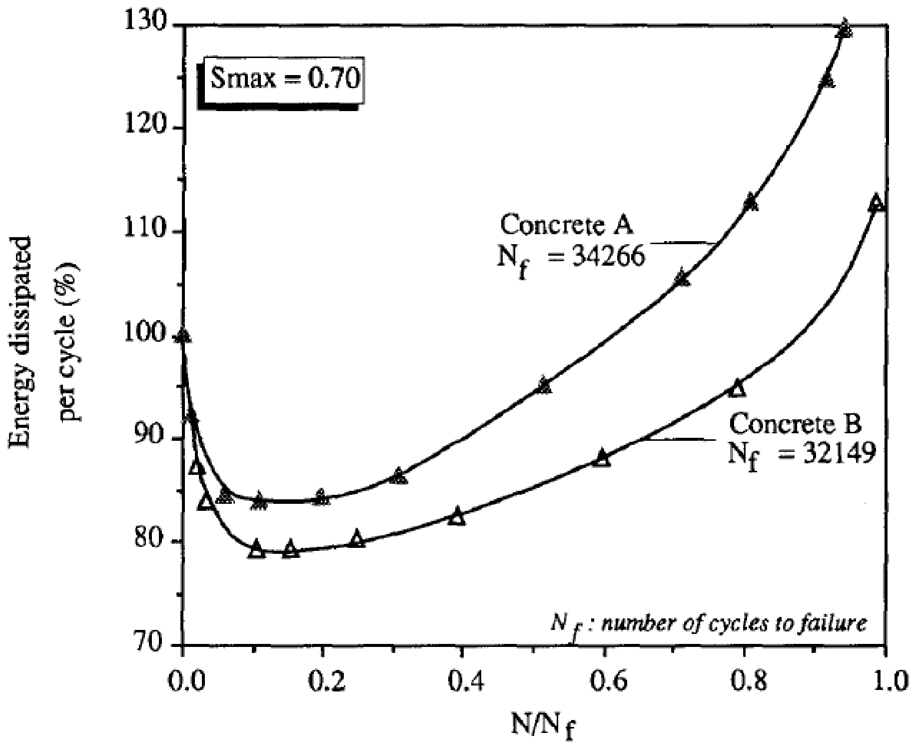

Figure 11: Comparison of Variation of Energy Dissipated with Life Ratio (Concrete A= 70MPa B=95MPa) (Do et al., 1993)

2.8 Literature Review Conclusion

Based on the literature presented in this section it can be seen that UTCRCP is a promising road paving technology that provides many desirable characteristics. Much research has been conducted into this system. However it is clear that increased knowledge into the components, and the interaction between these components, would assist in better prediction of the performance of UTCRCP. Improved performance prediction and optimisation would contribute to making this pavement technology more reliable and more cost effective.

Concrete, being the major constituent of UTCRCP, plays a major role in the performance of the pavement. From the work done to date it is evident that concrete of the required strength can be produced and placed in the tight dimensions required by UTCRCP. The influence of different compressive strengths on the mechanical performance, and fatigue life however does not appear to be well documented. Other aspects of the concrete that appear not to be well understood include the magnitude of thermal expansion that UTCRCP can experience resulting in buckling of the concrete. In implementation of experimental sections the effects of shrinkage also appear to have been ignored. The control of concrete shrinkage can be utilised to assist in preventing buckling of the pavement from thermal expansion but it should not be large enough to cause the reinforcing mesh to snap.

The spacing and size of the reinforcing mesh can be adjusted. Research to date appears to have found a solution that is both functional and constructible. The importance of using a ductile steel mesh is highlighted.

Steel fibres greatly enhance the flexural strength of concrete and are thus vital to UTCRCP. As demonstrated by the HVS tests the use of the incorrect fibre can lead to premature failure of the pavement. The effect of fibre content on the performance of UTCRCP however, does not seem to be well documented. The interaction between concrete strength and fibre content and the effect this has on the performance of UTCRCP is also a topic that has not yet been investigated in depth.

The nature of fibre reinforced concrete is such that the standard laboratory tests need to be modified in order to obtain meaningful results from these tests. Furthermore the evaluation of fatigue performance in the laboratory does not appear to be a common practice among designers of UTCRCP. Additional understanding of the advantages and limitations of laboratory tests used to evaluate fibre reinforced concrete would be beneficial to the development of UTCRCP as well as quality control during construction.

3 Experimental Design

3.1 Introduction

The purpose of these experiments was to evaluate the effect of both concrete strength and fibre content on the hardened properties of fibre reinforced concrete. Because only the hardened properties are of interest, tests were conducted after samples had cured for 28 days. No testing was conducted to monitor strength development during this time.

The range of concrete strengths and fibre contents selected to be tested resulted in 12 different combinations. In this document the term ‘batch’ will be used to describe each one of these combinations. All the samples of a batch were cast on the same day.

As fatigue performance was one of the key areas of interest, it was necessary to design the experiments to accommodate fatigue testing. Fatigue testing requires many load repetitions to be applied to a sample. Although the load applications were applied in quick succession by computer controlled apparatus, the vast number of load repetitions meant that the testing of one batch would take approximately five days. The casting and testing schedule was then planned with this in mind.

Concrete strength after 28 days is often used to classify concrete strength. Therefore it was selected to commence with testing the prepared samples 28 days after casting. When cement extenders are used significant strength gain can continue after 28 days. As cement extenders are used in UTCRCP concrete strength gain may continue during the 5 day test period. This may skew the test results in favour of the samples tested towards the end of the test period. In order to minimise this effect a number of measures were taken. The samples of each batch were tested in the same sequence, thus comparison of results between batches would not be affected by curing time. Furthermore, the majority of the tests were conducted within the first two days after the 28 days of curing had elapsed. As a further check, compressive and tensile tests were conducted at the beginning and end of the 5 days. These results were compared to check for the amount of strength gain over the period of testing. No evidence of strength gain during the 5 days was observed and therefore the average of all samples tested was used for all mixes.

3.2 Concrete Mix Design and Sample Preparation

Current UTCRCP design calls for the use of concrete with a characteristic strength of 90 MPa. Higher strength concrete is more costly. Not only are more costly materials required, but better quality assurance measures also need to be implemented. Higher strength concrete is generally less workable. This poses a challenge to the construction crew who must place and compact the concrete, resulting in increased cost of construction for the pavement. In order to determine the effect of concrete strength on the performance of UTCRCP three different strengths were selected; these were target strengths of 50, 70 and 90 MPa.

There are a number of different definitions of concrete compressive strength. Samples tested from any batch of concrete will yield a range of strengths. These will follow a normal distribution. Average compressive strength is the mean strength of the concrete. Characteristic strength is the strength that 95% of the concrete will exceed (Owens, 2009).

Increasing fibre content also increases the cost of UTCRCP. To determine the effect of increased fibre content on the UTCRCP performance, fibre content was also varied. Fibre contents of 50, 70 and 90 kg/m³ were selected for testing. Samples without fibres were also cast, and these were used as a control group. Because the interaction between concrete strength and fibre content is of interest, each concrete strength must be tested for every fibre content. Thus a total of 12 batches were cast. In order to reduce the number of variables, the cement used was all from the same batch from the supplier. Cement was stored carefully in the laboratory, however, in order to avoid possible aging of the cement skewing results, batches were cast in a random order. Target concrete strength, fibre content and casting sequence are shown in Table 2. The number and type of samples cast per batch is indicated in Table 3.

Table 2: Experimental Design: Showing Casting Sequence, Fibre Content and Target Concrete Strengths.

		Target Concrete strength (MPa)		
		90	**70**	**50**
Fibre content (kg/m³)	**90**	B1	B10	B3
	70	B6	B9	B12
	50	B11	B8	B5
	0	B4	B7	B2

Initially a number of trial mixes were prepared in order to refine the consistency of the mix and to determine the compressive strength to water cement ratio relationship. The relationship determined is shown in Figure 12. This relationship was used to select the water cement ratio that would give the desired compressive strength. The mix design proportions used are given in

Table 4. The selection of water cement ratio was done on the basis of target mean strength and not on characteristic strength. Steel fibres were added to meet the required fibre content and no adjustments were made in the mix design to account for different steel fibre contents. The error caused by this was deemed to be negligible in a comparative study.

Table 3: Number and Type of Samples Cast Per Batch

Sample	Number Cast	Test
 100mm Cube	6	Compressive strength
 Cylinder Height: 300 mm Diameter: 150 mm	7	4 x Splitting cylinder strength 3 x Young's Modulus and Poisson's ratio
Slab  700 mm x 250mm x 55 mm	8	3 x Monotonic Loading 2x Precrack 1x 10 000 load cycles 1x 100 000 load cycles 1x 1 000 000 load cycles

For the sake of completeness the properties of the concrete constituents were measured. Grading curves for the sand and stone are presented in Figure 13. Particle size distribution was measured using a combination of sieve analysis for particle sizes greater than 600 µm, and an optical particle sizing instrument (Malvern Mastersizer 2000) for the fraction smaller than 600 µm. Relative density was measured using a helium gas pycnometer. The results are given in

Table 4 with the mix designs.

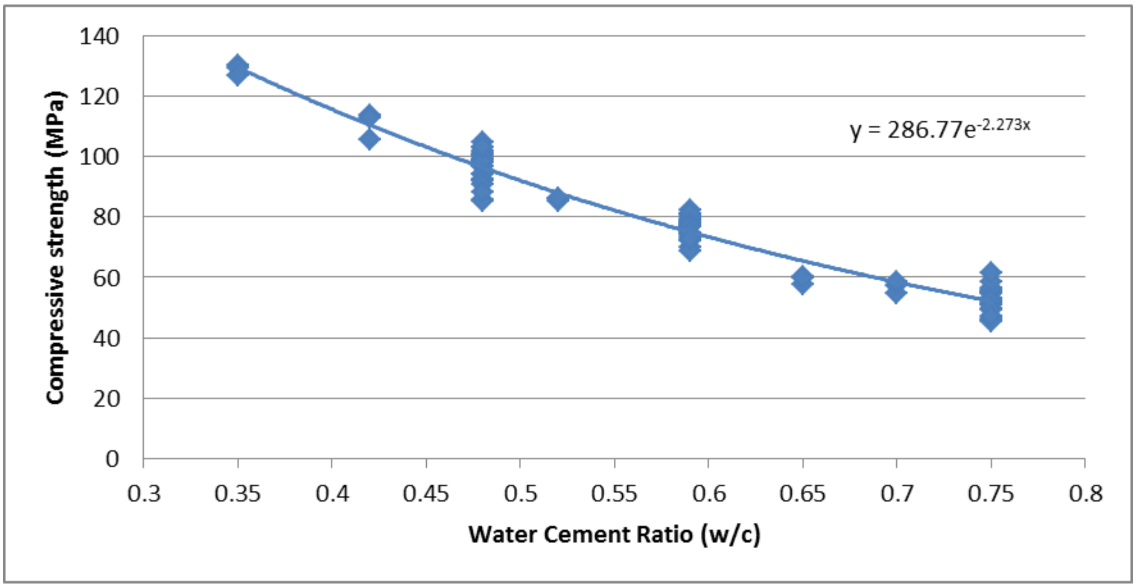

Figure 12: Compressive Strength vs Water Cement Ratio Relationship

Table 4: Concrete Mix Designs

	Unit	**Relative Density**	**Desired Concrete Mean Strength**		
			90 MPa	**70 MPa**	**50 MPa**
Water cement ratio			0.48	0.59	0.75
Cement (CEM1 52.5N)	kg/m^3	3.135	267.8	217.9	171.4
Fly Ash	kg/m^3	2.263	45.07	36.66	28.84
Condensed Silica Fume	kg/m^3	2.319	22.53	18.33	14.42
Water	kg/m^3	1.000	161.0	161.0	161.0
Dolomite 9.5 mm aggregate	kg/m^3	2.823	759.4	759.4	759.4
Dolomite sand	kg/m^3	2.840	1268	1333	1393
Chryso Premia 310	l/m^3	1.049	5.92	4.82	3.79
Chryso Antifoam	ml/m^3	0.806	0.5	0.5	0.5
Polypropylene fibre	kg/m^3	0.915	2	2	2
Steel fibre (30 mm 0.5 mm diameter hooked end cold drawn wire fibres.)	kg/m^3	7.675	0,50,70,90	0,50,70,90	0,50,70,90

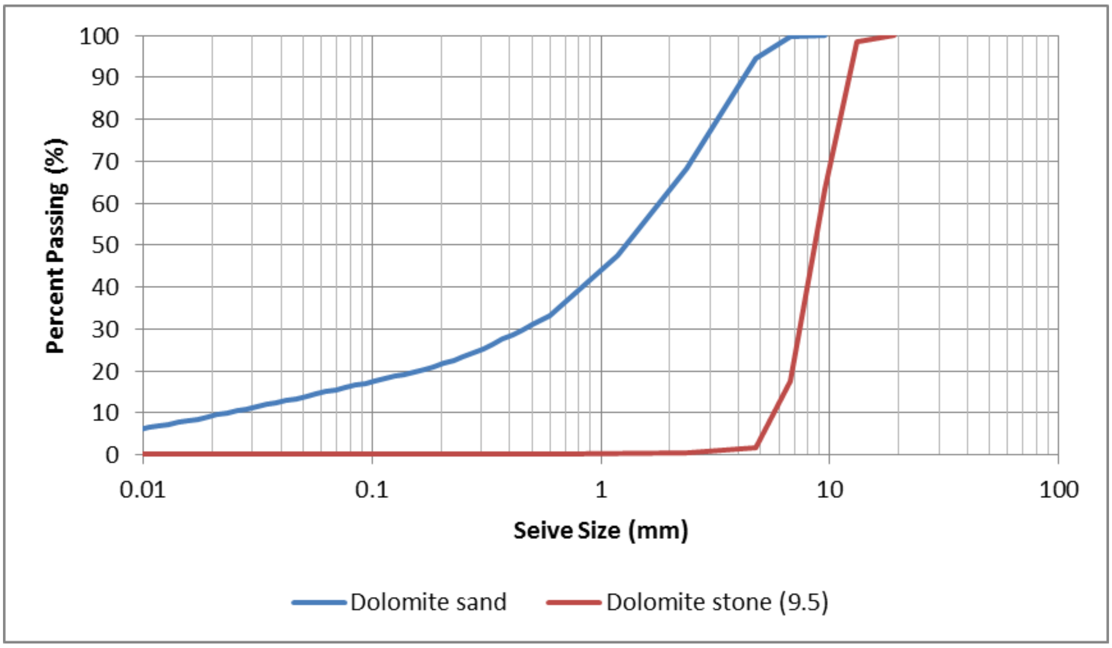

Figure 13: Sand and Stone Grading Curves

The concrete was mixed in a pan mixer. Due to the limitation in capacity of this mixer two mixes had to be batched and mixed separately to cast one full set of samples. The dry constituents, excluding the steel fibres, were added to the mixer and the mixer was started. The Superplasticiser and de-air-entrainer were then added to the water. The liquid was then added into the mixer. In order to give adequate time for the condensed silica fumes to thoroughly distribute in the concrete, the concrete was mixed for 10 minutes. Toward the end of the mixing, the steel fibres were added. In order to prevent the steel fibres from clumping in the mixer they were scattered by hand into the mixer.

Concrete was then placed into moulds and compacted by means of a vibrating table. Moulds were placed in a temperature and humidity controlled room overnight. The samples were then de-moulded and placed into curing tanks with a constant water temperature of 24° C for 28 days.

Adding fibres to concrete greatly decreases the workability, particularly at the high fibre contents used in UTCRCP. Low water cement ratio also is not favourable to good concrete workability. This necessitated the use of superplasticisers in this concrete. In previous work done it was found that high strength fibre reinforced concrete, even when well compacted, could contain high percentages of air. To mitigate this possible effect de-air-entrainer was also used in the mix. On some of the first mixes a slump test was conducted to measure workability. Negligible slump was observed. There are other workability test methods available. However, for the purposes of this study, workability was not an area of focus and workability was not recorded. Despite this, it can be noted that the mix could be handled and compacted without excessive difficulty.

3.3 Compressive Strength and Compaction Measurements

In order to measure the compressive strength of the concrete the standard cube test was used, SANS 5863 (SABS, 1994). 100 mm cubes were used. Three cubes were tested 28 days after casting. Upon the completion of the fatigue testing a further three cubes were tested to determine if significant strength gain had occurred in this time.

A further quality parameter of the concrete, that can be determined using cubes, is air content. This gives an indication of how well compacted the concrete is. In general it has been found that for every 1% air content there is approximately a 6% reduction in compressive strength (Illstone and Domone, 2008). Before crushing, cubes are weighed in air and then submerged in water. From this the volume of the cube can be determined which can be compared to the theoretical density of the concrete calculated from the mix design proportions. An air content can be determined using Equation 3.1 and Equation 3.2.

$$Measured\ Density = \frac{mass\ in\ air}{mass\ in\ air - mass\ in\ water} \times 1000 \qquad \text{(Equation 3.1)}$$

$$\%air = \frac{Theoretical\ density - Measured\ density}{Theoretical\ density} \times 100 \qquad \text{(Equation 3.2)}$$

3.4 Tensile Strength Measurements

The indirect tensile strength of the concrete was determined by using the splitting cylinder test. This is a standard concrete test, the method is described and specified in ASTM C496/C496M-04 (ASTM, 2009a) and SABS 6253 (SABS, 1994). For the purpose of measuring the post crack behaviour of the concrete, horizontal deflection of the sample was recorded using Linear Variable Differential Transformer (LVDT) attached to metal pins that were inserted into holes that were drilled into the sample. Two LVDTs were used, one at either end of the sample. This method was described by Denneman et al. (2011) and Denneman et al. (2012).

Samples were placed in a loading rig with hard board strips on the contact surfaces and loaded until failure, or until the crack had opened wider than 2 mm, the maximum range of the LVDTs. Figure 14 shows the typical configuration of a splitting cylinder test conducted during these experiments.

The tensile stress induced by the applied load can be calculated using Equation 3.3.

$$f_{st} = \frac{2P}{\pi l d} \qquad \text{(Equation 3.3)}$$

Where:
f_{st} is the tensile stress (MPa)
P is the load (N)
l is the length of the specimen (mm)
d is the diameter (mm)

This is the formula recommended for use by both the SABS (1994) and ASTM (2009a). The above equation is derived from Boussinesq theory and was initially solved by Hertz (Timoshenko & Goodier, 1951). The assumption made is that the load is applied as a line load. In practice however the load is applied over finite width. In order to take this effect into account Tang (1994) suggested the use of the modification indicated in Equation 3.4.

$$f_t = \frac{2P}{\pi l D}\left[1 - \left(\frac{b}{D}\right)^2\right]^{\frac{2}{3}} \qquad \text{(Equation 3.4)}$$

Where:
f_t is the tensile stress (MPa)
P is the load (N)
l is the length of the specimen (mm)
D is the diameter (mm)
b is the width of the load strip (mm)

The formula with the adjustment for load width has been successfully used by other researchers when numerically modelling the fracture behaviour of fibre reinforced concrete (Denneman, 2012). The formula with the adjustment will thus be used to calculate tensile stress for the splitting cylinders.

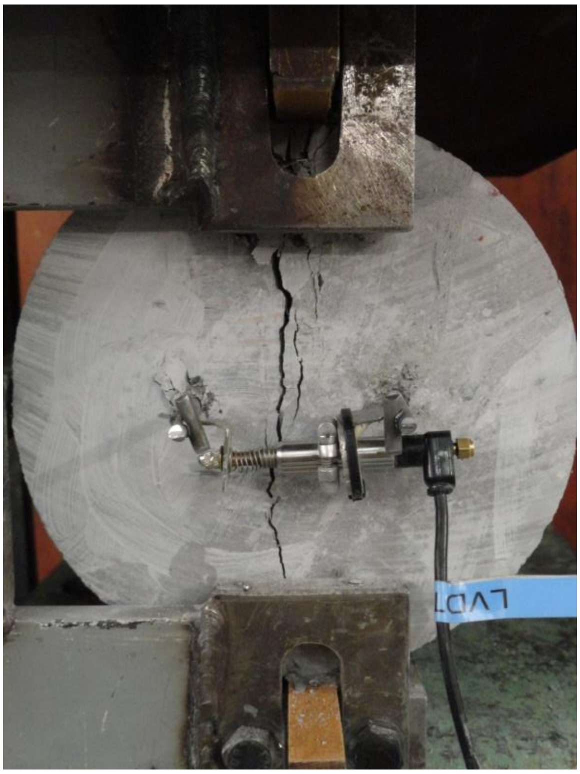

Figure 14: Splitting Cylinder Test

Tensile strain measurements were calculated by using the average displacement measured by the two LVDTs, with the 50 mm distance between the metal pegs used as the base length. Load was applied to the specimen until the horizontal movement of the pegs had exceeded 2 mm.

The result of a typical splitting cylinder test is shown in Figure 15. Points that will be used as comparative measures in the results are indicated on the figure.

- First Crack Stress: this is taken as the stress at which the stress strain relationship ceases to be linear (Johnston and Zemp, 1991; ASTM C1018-97, 1997)
- Principal Crack Stress: the peak or plateau stress following shortly after the First crack strength.
- Maximum Stress: the maximum stress resisted by the sample

Both First crack strength and principal crack stress were identified manually for each sample by inspecting the stress strain plots. Maximum stress was identified automatically.

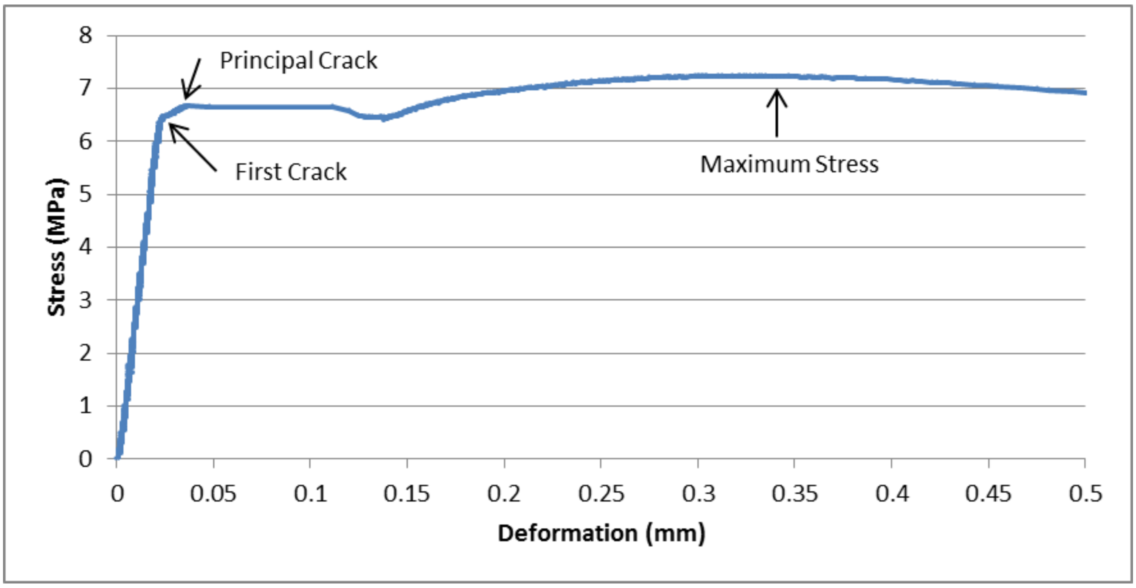

Figure 15: Splitting Cylinder Test Result for a Sample of 90 MPa Concrete and 90 kg/m³ Fibre

The standard splitting cylinder test is used to determine the principal crack stress. However, in order to more fully understand the material characteristics, it is also necessary to quantify the post crack performance of the concrete. Strain energy density makes for a good comparison but does require the measurement of deformation in the test.

Strain energy is the energy absorbed by a material due to deformation. Strain energy density is defined as the strain energy per unit volume; this can be calculated as the area under the stress strain curve. Strain energy density has the unit Joules per cubic meter (J/m³) and it can be calculated using Equation 3.5:

$$u_0 = \int_0^{\varepsilon} \sigma \, d\varepsilon \qquad \text{(Equation 3.5)}$$

Where:
u_0 is Strain energy density (J/m³)
σ is stress in Pa
ε is strain

The integration limits were selected to represent strain energy for a range of deformations. The smallest deformation chosen was 0.05 mm in order to quantify the performance at small deflections. Other strain limits were selected to represent crack widths of 0.1 mm, 0.25 mm, 0.5mm, 0.75 mm and 1 mm.

3.5 Young’s Modulus & Poisson’s Ratio

Young’s Modulus and Poisson’s ratio are important material properties used as input into a design. These parameters are measured from tests performed on concrete cylinders with a diameter of 150 mm and a height of 300 mm. An example of a specimen in the loading press with the measurement equipment attached is shown in Figure 16. Before testing, the rough top surface of the cylinder was milled smooth. Samples were tested shortly after removal from the curing bath and were therefore in a wet state.

The ASTM C649-02 test procedure was followed (ASTM, 2009b). For this method it is standard practice that two points are recorded per load cycle, the stress at a strain of 50 microstrain and the strain at 40% of the ultimate strength of the concrete. The stiffness is then calculated as the gradient between these two points. This is referred to as the chord modulus of elasticity. Similarly the Poisson’s ratio is calculated using these two points. The assumption inherent in the above calculation is that the concrete will behave in a linear elastic manner within this load range. In addition to these points, interim readings were taken in order to more fully capture the material behaviour. Readings were recorded every 50 kN (2.83 MPa).

Figure 16: Young’s Modulus and Poisson’s Ratio Test Equipment

As per the test specification the specimen was loaded to 40% of the ultimate load and then unloaded. No readings were recorded during this cycle. The purpose of this is to allow for bedding of the specimen in the press and to allow for seating of the gauges. The 40% load was then reapplied with measurements being recorded during loading.

Two further load cycles were then applied to the specimen; a load of 70% of the ultimate load was applied in order to investigate material behaviour at higher load levels. It was not possible to test to failure of the sample as this would risk damage to the measurement equipment. Three samples were tested per batch and the results of each were recorded. Where possible the mean of the results was used.

3.6 Flexural Performance – Under Monotonic Loading

A flexible pavement structure transfers force by bending. The laboratory tests discussed up to this point measure material parameters that influence the performance of the pavement structure but do not evaluate the flexural performance of the material directly. While no laboratory test can fully replicate all the intricate interactions that a full pavement structure will be subjected to, testing the flexural capacity will give a good indication of the performance that can be expected.

Flexural or beam tests are frequently used as an indirect measure of the tensile strength of the concrete. This test uses a concrete prism of 100 mm by 100 mm spanning 300 mm, loaded in 4 point bending (ASTM, 2004). Using elastic theory the tensile strength of the concrete is determined.

Denneman (2010) showed that fibre reinforced concrete exhibits a strong size effect due to its high post crack tensile capacity. The implication of this is that in order to properly measure the performance of a fibre reinforced concrete the laboratory specimen must be of the same depth as the constructed height. Flexural tests were thus conducted on slab samples that have the same thickness as the UTCRCP. Samples are prepared by casting the prepared concrete into a mould. The final dimensions of the samples were 700 mm by 250 mm with a height of 55 mm. The thickness of the sample is the same as the constructed thickness of a UTCRCP layer. Steel reinforcing, in line with current design, was also placed in these samples. This is a reinforcing mesh of 5.6 mm diameter high yield steel with a spacing of 50 mm centre to centre.

After curing, the samples were tested in four point bending in the apparatus shown in Figure 17. The bottom supports are at a distance of 450 mm and the top two Rollers are 150 mm apart. Deflection is recorded by two LVDTs in the centre of the slab. Each LVDT rests on the surface of the slab approximately 10 mm from the edge of the slab. The cylinder head deflection is also recorded using an LVDT.

The slabs are tested in flexure. This is similar to the loading that the UTCRCP will experience when constructed as a pavement. Flexural tests in the laboratory can be conducted much more cheaply and quickly than HVS testing, without the variables added by changing layer works conditions. However this test does have limitations that can only be avoided by conducting full scale accelerated pavement tests. Flexural performance was tested under two different loading regimes; these were monotonic and cyclic loading.

The results of these tests will be used to calculate the mechanical properties of the fibre reinforced concrete in the UTCRCP system. The intention is to observe the interaction, if any, between the different variables. Conclusions regarding the material's performance as an UTCRCP layer can be drawn by comparing the results of the tested slabs.

Figure 17: Slab Testing Apparatus

Monotonic loading testing refers to the tests where the sample is loaded by an increasing load until failure. Samples were tested in deflection control. In deflection control load is increased at a rate that results in a constant increase in deflection of 0.025 mm per second until failure of the slab or a maximum central deflection of 25 mm. During the test load and deflection data was recorded at a rate of 100 Hz.

Deflection of the slab was measured during testing. Due to the nature of failure of the concrete it was found that a single measurement of deflection was not effective. Two LVDTs resting on the top surface of the concrete in the centre of the span give good deflection measurements at low deflections.

The measurements of these two were averaged to account for any twisting of the specimen. At high deflections the measurements of these two instruments can be influenced by crushing concrete on the top surface of the slab (i.e. the slab did not crack at the midpoint between the two rollers). At low deflections the cylinder head deflection does not capture the linear load deflection relationship because during initial contact the rollers bed into the concrete. Because the linear load deflection relationship is an indication of stiffness, measuring this linear relationship is important. Thus where deflection information is given for slabs under monotonic loads average deflection of the two side LVDTs is used up to a deflection of 1 mm upon which the cylinder head deflection is then used for the rest of the test.

A number of different performance measures can be extracted from the data recorded during the loading of samples. An example of a typical result for this kind of test is show in Figure 18.

The principal crack load indicates the point where the tensile stresses in the concrete exceed the concrete tensile capacity. At this point the concrete matrix cracks, the steel reinforcing bars and steel fibres are then activated and further carry the load. At this point there is a sudden decrease in gradient from the preceding linear elastic section. In a sample with no reinforcing and no steel fibres the principal crack load is equal to the maximum load. The principal crack load is sensitive to the depth of the sample. Even if samples are cast with care, some variation in depth will exist. It is therefore a better practice to correct for this variation in depth by converting the principal crack load to a stress using Equation 3.6. The result is equivalent to the Modulus of Rupture (MOR) of un-reinforced concrete. As it is an indirect measure of the tensile strength of the concrete, the results can be compared with the tensile strength of the splitting cylinder test.

$$f_t = \frac{PL}{bd^2} \qquad \text{(Equation 3.6)}$$

Where:

f_t is the tensile stress (MPa)

P is the load (N)

L is the distance between the supports (mm)

d is the depth of the specimen (mm)

b is the width of the specimen (mm)

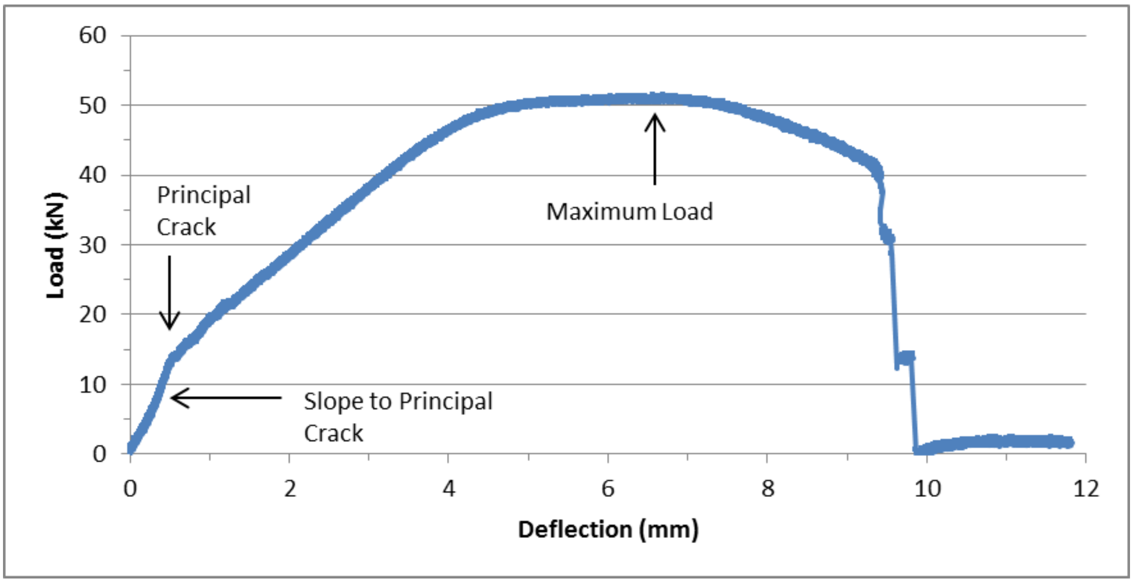

Figure 18: Typical Load Deflection Plot for Monotonically Loaded Slab Test (Concrete 70 MPa, Fibre Content 70 kg/m³)

Further information that can be extracted from the flexure test results is the slope to principal crack. This represents the stiffness of the un-cracked concrete section. At this point the reinforcing steel and steel fibres have not been fully activated, thus it should correlate well with the measured Young's Modulus of the concrete. This slope is an indication of the behaviour and condition of the material at low flexural deflections. This slope is also used as a base line point to quantify the damage done during cyclic loading, which will be discussed in more detail in Section 3.7.3. The slope is calculated by linear regression of the recorded load deflection data points up to the principal crack load.

The maximum load carried by the sample is also recorded. Because the samples are tested in deflection control the maximum load does not necessarily occur at the point of failure. The maximum load carried is extracted from the recorded load deflection data.

The final parameter extracted from the monotonic loading tests was Energy absorption. Energy absorption of the sample is equivalent to the work exerted on the sample from the loading arm. This is equivalent to the area under the load deflection curve (Japanese Concrete Institute, 1983).

One must however account for the bedding of the loading apparatus into the concrete sample. Depending on the moment at which the data recording is started, the energy error accumulated during the bedding in of the loading apparatus can be significant. To overcome this challenge the following procedure was used. It was found that the average principal crack occurred at a deflection of 0.497 mm with a standard deviation of 0.0459. The 5th percentile principal crack deflection was calculated to be 0.42 mm. It can be noticed that the first part of the loading deflection curve is approximately linear (Figure 18). Therefore the energy absorption for deflections below 0.42 mm was calculated using the slope to principal crack. For values above this deflection the area was calculated by numerical integration of the recorded load deflection data and using the midpoint rule.

Energy absorption must be calculated up to a specific deflection. The deflections at which energy absorption was determined are; 1.8 mm (representing the serviceability limit ratio of span/250), 3 mm, 5 mm, 10 mm, 15 mm, 20 mm and 25 mm.

In summary, a number of performance measures can be extracted from slabs tested under monotonic loading. These include: principal crack load, MOR, slope to principal crack, maximum load and energy absorption.

3.7 Flexural Performance – Cyclic Loading

Monotonic loading is often used in material testing in order to determine the strength limits of a material. The nature of loading of road pavements is however cyclic in nature. In order to get an indication of the performance of the fibre reinforced concrete as a road pavement it is necessary to also measure the material under cyclic loading.

The cyclic testing regime included a number of phases. First samples were precracked to mimic the operational condition of the pavement. Secondly cyclic loading was applied. Lastly the sample was tested under monotonic loading to failure. As a base line some samples were tested under monotonic loading after precracking without applying cyclic loading. It is anticipated that by comparing the monotonic load test results of slabs that were not precracked slabs, with those that were precracked and slabs that were precracked and subjected to monotonic loading, the damage accumulation in the concrete matrix can be measured. This process is demonstrated graphically in Figure 19.

From each batch, a total of 6 slabs were cast. Of these slabs, two were tested in monotonic loading, one was precracked and then monotonically loaded to failure without cyclic loading being applied. The remaining slabs were precracked, cyclic loading was then applied after which the sample was monotonically loaded to failure.

In the subsequent paragraphs of this section details of each activity in this testing regime is presented.

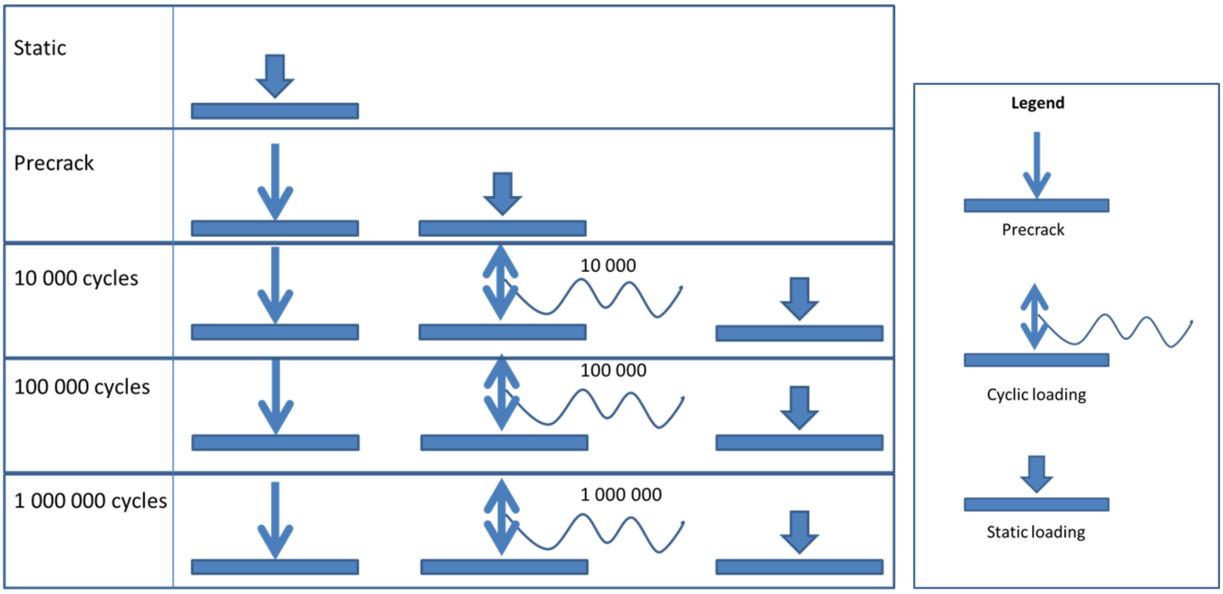

Figure 19: Slab Loading Sequences

3.7.1 Precracking

The design philosophy of a continuously reinforced concrete pavement is that the concrete will crack but the road remains impervious to water because these cracks are held tightly closed by the steel reinforcing and fibres. Similar to expansion and contraction joints these small cracks also allow for the volumetric change of the material caused by environmental factors such as temperature and moisture. Cracking may occur early in the pavements life due to load application, temperature change or shrinkage of concrete during curing (Perrie and Rossmann, 2009; de Larrad, 2005).

Because the pavement material will serve most of its life in a cracked state the cracked fatigue behaviour is of more interest than the un-cracked fatigue behaviour. In order to crack the material the slab was loaded in deflection control at a rate 0.025 mm/min to a load exceeding 19 kN. Once this load was reached the load was released. In previous research 19 kN was found to represent the 99th percentile MOR load for slabs containing similar strength concrete. The statistics of the maximum loads applied to the precracking samples is given in Table 5 Precraking Load StatisticsTable 5. The values in this table indicate that all precraked samples were exposed to very similar loading during the precracking procedure.

Table 5 Precraking Load Statistics

Minimum	21.09 kN
Maximum	23.72 kN
Mean	21.91 kN
Standard deviation	0.46 kN

During pre-cracking the deflection data was recorded at a frequency of 100 Hz. From this data the true MOR and slope to MOR could be determined. Typical pre-cracking results are shown in Figure 20, the points that represent the MOR and slope to MOR are marked. Also shown is the unloading of the sample. During loading the crack was not easily visible and after unloading the crack closed up and was not visible.

In order to quantify the loss in strength caused by fatigue loading the stiffness of samples was measured after applying different numbers of load repetitions. These can be compared to the un-cracked stiffness during the first loading. The stiffness is calculated as the slope of the linear regression line up to the MOR point. Modulus of rupture is the point where the concrete cracks and the steel reinforcing must then carry the tensile forces in the beam. It is identifiable by the point where there is a sudden decrease in gradient from the preceding linear elastic section.

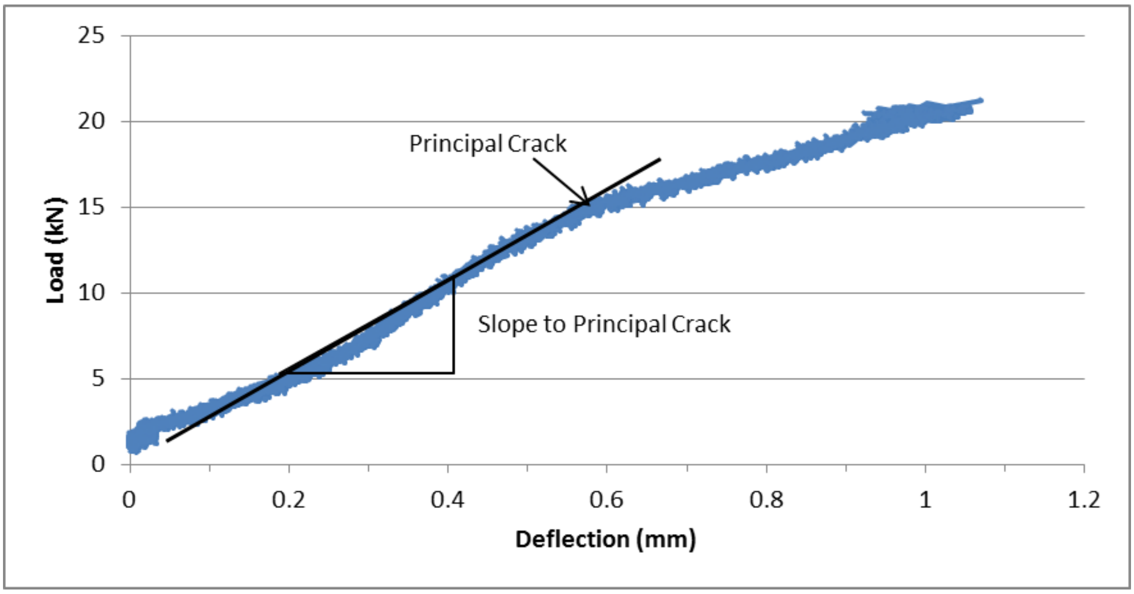

Figure 20: Pre-crack Load Deflection Results (90 MPa, 90 kg/m³)

3.7.2 Cyclic Loading

After pre cracking the cyclic load is applied to the sample for the required number of repetitions. The load is applied in a sinusoidal manner at a frequency of 3 Hz. During application of the cyclic load data is recorded every 500 cycles for 5 cycles at 500 Hz.

If the sample reached the required number of load cycles without failing the sample is then tested monotonically until failure. The decrease in stiffness with increasing load repetitions can then be compared. The load repetitions selected for the cyclic loading were 10 000, 100 000 and 1000 000 cycles.

The load cycles were the same for all the slabs. This load was determined by calculating the stress the UTCRCP would experience in a road structure under an 80 kN axle load. Using the computer software, Chev 15, the stress at the base of the concrete was calculated. This calculation is based on linear elastic theory. Material stiffness's were assumed from the values recommended by Theyse et al. (1996). The materials and dimensions of the layer works assumed in the calculations were similar to that of the UTCRCP pavement constructed on of the R104 experimental section. Maximum horizontal stress at the base of the UTCRCP layer was calculated to be 1.648 MPa in tension. The load needed to apply this stress was then determined to be 2.769 kN.

When fatigue testing is performed on asphalt the rate of load application is critical. The mechanical behaviour of Asphalt is highly temperature dependent - should the loading rate be too fast the test sample may build up heat. This will weaken the sample and skew the results. Concrete is not considered to be as temperature sensitive as asphalt. However, it was undesirable for an excessively fast load application rate to cause heat build-up in the sample.

In order to determine the realistic range of testing frequencies calculations were conducted assuming different vehicle speeds of trucks. The loading frequency was determined using typical truck axle spacing. In this case the dimensions used were 1.24 m as the axle spacing on a dual axle and 3.73 m between steer and drive axles (O'Leary, 2009). While these dimensions are not fixed and will vary they will give a representative idea of the values involved. Loading frequency was taken as the full cycle between loading and unloading. It was assumed that both tyres have the same tyre inflation pressure and same load, the spacing of the tyre contact areas will be the same as the spacing of the axle. It was assumed that the smallest following distance would be one vehicle length. The loading frequencies determined are shown in Figure 21.

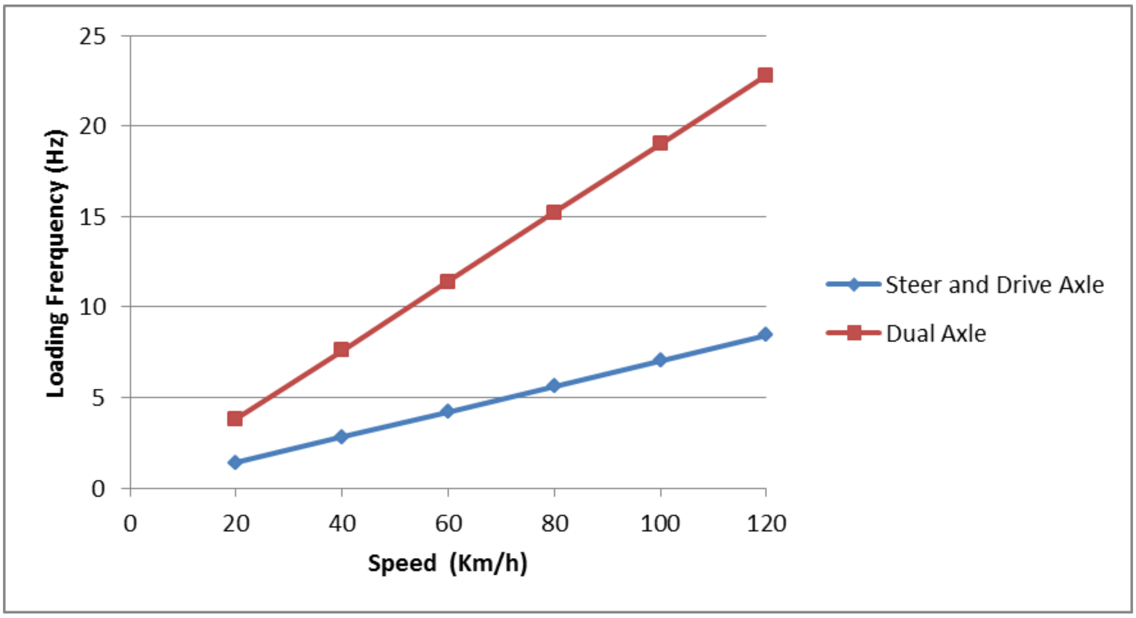

Figure 21: Loading Frequencies for Different Wheel Spacing's and Speeds

The loading apparatus used to test the samples could function at a maximum of 5 Hz and inspecting Figure 21 this rate is well within the anticipated maximum pavement loading frequency. A loading frequency of 3 Hz was selected. During testing a thermal camera was also used to check for heat build-up in the sample. However, no heat build-up was detected.

3.7.3 Processing Cyclic Data and Performance Measures

The application of 1 million load cycles at a frequency of 3 Hz takes over 92 hours. Recording the load deflection data for the entire duration of the test would result in very large data files that would be difficult to work with or interpret.

In order to capture the fatigue performance of the slab during the test, it was decided to capture a snapshot of the performance at regular intervals. In this way the performance with time can be followed. Due to the fast cycling of the load the logging rate of the data recorded during these intervals had to be increased. During application of the cyclic loading data was recorded every 500 cycles for 5 cycles at 500 Hz. In this way, regular snapshots of the condition of the sample can be obtained. Damage can then be tracked and the performance of the different concrete strength and fibre contents could be compared.

From the five loading and unloading cycles recorded the following parameters were extracted: stiffness, loop width, minimum deflection and maximum deflection. The significance and calculation methodology of each is detailed in the sections below.

Minimum and maximum deflections give an indication of the damage to the slab. Increasing maximum deflection indicates a weakening of the slab structure under loading while minimum deflection tracks the amount of permanent deflection of the slab. Minimum and maximum deflection could be extracted from the data as the minimum or maximum deflection recorded during any of the 5 recorded load cycles.

Stiffness is a measure of the relation between load and deflection of the slab. It is calculated as the gradient of the linear relationship between the load and deflection data, as depicted in Figure 22. The interpretation on this parameter is that a higher stiffness indicates that less deflection occurs for the same quantity of load. And conversely, a lower stiffness would indicate that more deflection occurs for the same quantity of load. This stiffness would therefore be directly proportional to the load distributing capacity of the pavement slab. Stiffness is determined by the best fit linear regression of all 5 loading cycles. This is calculated using the standard linear regression function available in Excel or GNU Octave (Eaton et al., 2014) with the load and deflection data points as inputs.

Hysteresis behaviour can be used to track the damage of samples under cyclic loading. In order to quantify the amount of hysteresis observed the width of the hysteresis loop was determined. The loop width is shown in Figure 22, and is calculated by the following method (see Figure 23): The perpendicular distance between each data point and the best fit linear regression line is calculated by using Equation 3.7. The arithmetic mean and standard deviation of all the distances are calculated. The 2nd and 98th percentile distance are then calculated using the mean and standard deviation and assuming a normal distribution. The loop width is defined as the difference between the 2nd and 98th percentile distance.

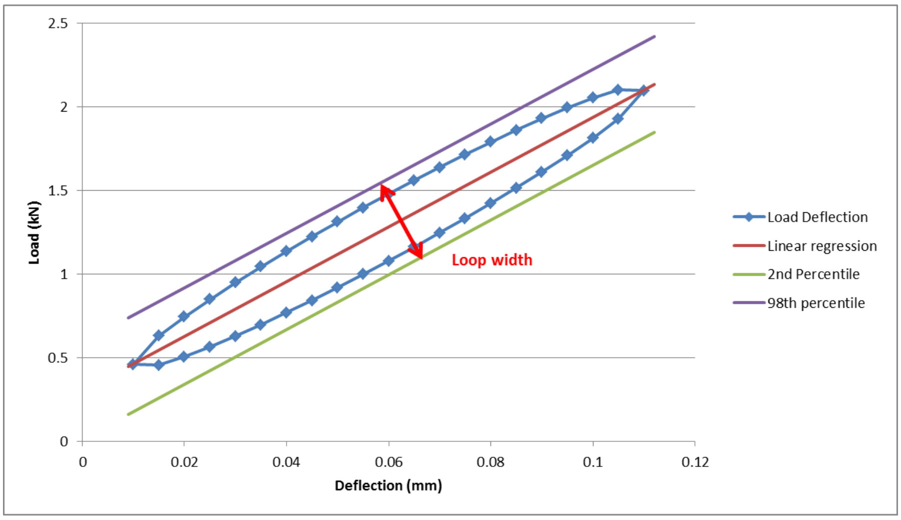

Figure 22: Theoretical Cyclic Loading Behaviour with Demonstration of Performance Measures

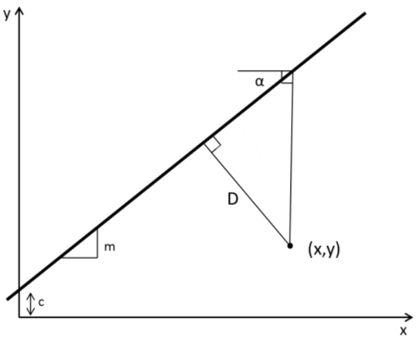

Figure 23: Determination of Perpendicular Distance of Data Point from Linear Regression Line

$$D = (mx + c - y)\,\mathrm{SIN}(90 - \mathrm{TAN}^{-1}(m)) \qquad \text{(Equation 3.7)}$$

Where:
D is the perpendicular distance of a data point to the best fit linear regression line
m is the gradient of the best fit linear regression line
c is the intercept of the best fit linear regression line with the y axis

Hysteresis energy can also be directly evaluated. The loading portion of the loop represents work done by the apparatus on the sample. The unloading portion of the loop represents work done by the sample on the loading apparatus. The area of the loop is thus irrecoverable mechanical work (Erber et al., 1993). This energy is calculated in Joules using the recorded load deflection data. The average energy of the 5 cycles recorded is used for each data point.

Maximum deflection and minimum deflection, stiffness, and loop width were determined for all the samples. This was used to track the fibre reinforced concrete performance with the application of cyclic loading, as well as the performance across the various mix combinations.

4 Results and Discussion

4.1 Introduction

In this chapter the results of the various tests conducted as part of the experiment are presented and discussed. The implications and relevance of the results are also dealt with in this section. While describing the behaviour of the same concrete, different parameters and tests are dealt with individually to understand the influence of each parameter more clearly. The complex interactions of fibre reinforced concrete cannot be overemphasized – where a sample may perform well in one test, it may perform poorly in another.

4.2 Compressive Strength Results

Compressive strength is one of the most vital parameters for concrete. Thus it has become the most commonly performed test in terms of construction quality control. The average compressive strength measured for the various batches are presented in this section.

The compressive strength results are shown in Table 6. In all cases the concrete strengths obtained are acceptably close to the target strength. The average compressive strength is used to plot the relationship between strength and other parameters.

In addition to the average compressive strength, presented in Table 6, Standard deviation and Characteristic strength are also reported. The Standard deviation can be used by future producers of UTCRCP to adjust the mix design for the target strength they require. As a reference characteristic strength for the mixes is also displayed. Characteristic strength is defined as the strength that 95% of samples will exceed (Owens, 2009).

Table 6: 28 Day Compressive Strength Results

		Average Compressive strength (MPa)			Mix Standard Deviation			Characteristic strength (MPa)		
Desired Compressive strength (MPa)		**90**	**70**	**50**	**90**	**70**	**50**	**90**	**70**	**50**
Fibre content (kg/m³)	**90**	101	78	54	3.08	1.64	1.86	95	75	50
	70	95	80	57	2.77	1.70	2.48	90	77	53
	50	100	76	52	1.73	1.61	0.76	98	74	51
	0	89	72	50	3.63	2.19	1.77	83	68	47

The results of the cubes tested at the start and end of the testing period are shown in Figure 24. In all cases it was found that no significant change in strength had occurred over the testing period. Results can therefore be compared without needing to take change in strength over the testing period into account. Because no significant gain in strength was recorded the average of the compressive strength of all the cubes was used as the compressive strength.

The air content measurements are presented in Table 7. It should be noted that the air content shown is calculated as a percentage of the theoretical maximum density and thus some of the small values are negative. The results indicate that the concrete was well compacted.

Table 7: Air Content (%) For The Different Concrete Mixes

		Compressive Strength (MPa)		
		90	**70**	**50**
Fibre Content (kg/m^3)	**90**	0.1	0.3	0.6
	70	0.3	-0.3	0.2
	50	-0.3	0.5	1.2
	0	0.0	1.5	1.4

The effect of fibre content on compressive strength of concrete is shown in Figure 25. It is observed that fibre content has a minimal effect on compressive strength even at high fibre contents. This is in line with the findings of Illstone and Domone (2008) as well as Song and Hwang (2004). Thus the addition of steel fibres can be expected to have little effect on the compressive strength of concrete.

Although cube testing is a vital quality control test and should be used during the construction of UTCRCP, engineers must be aware that the results of compressive strength do not give any indication as to the quantity or effectiveness of the fibres.

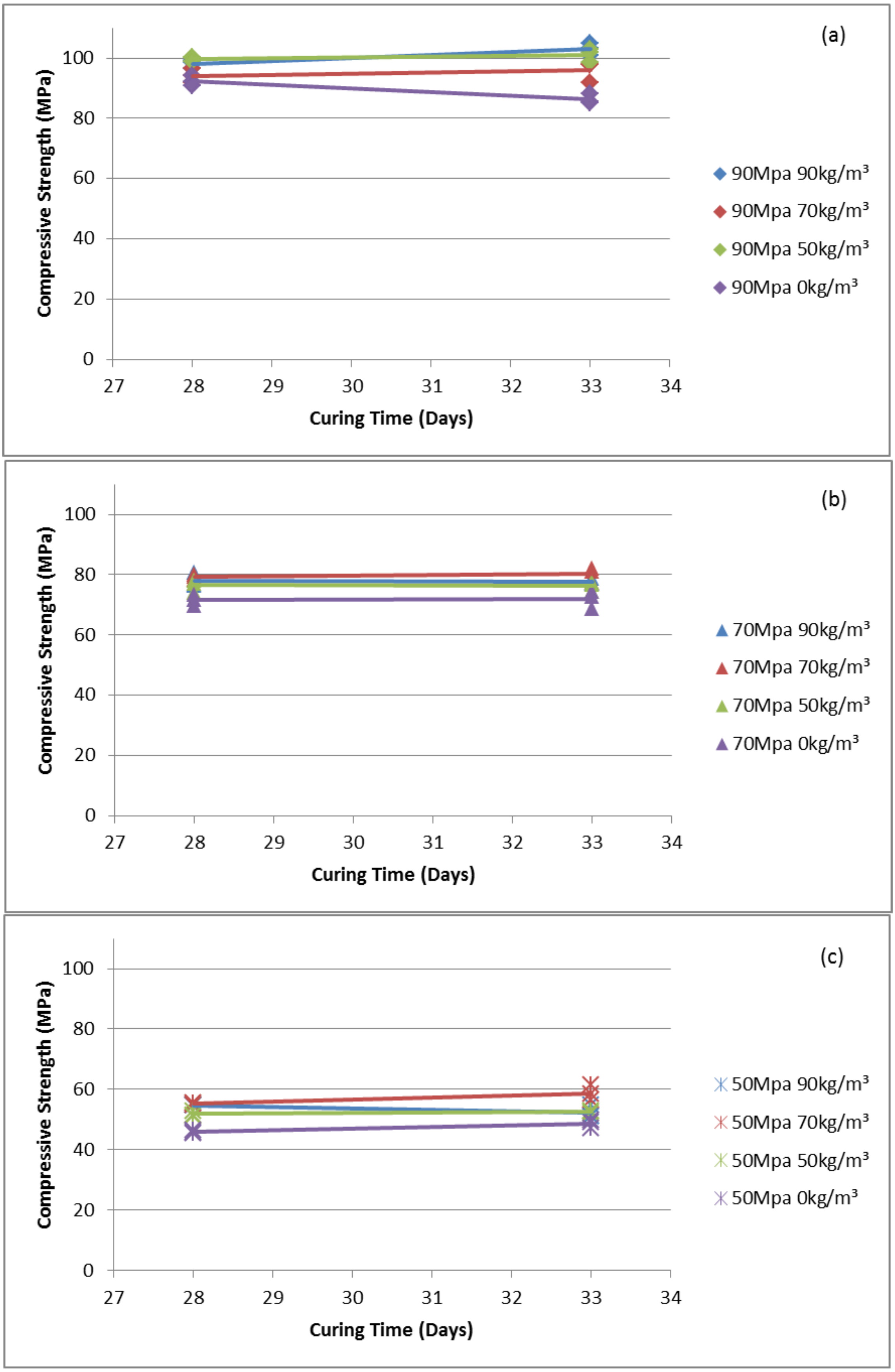

Figure 24: Compressive Strength Gain Over The 5 Day Testing Period: (A) 90 MPa Samples (B) 70 MPa Samples (C) 50 MPa Samples

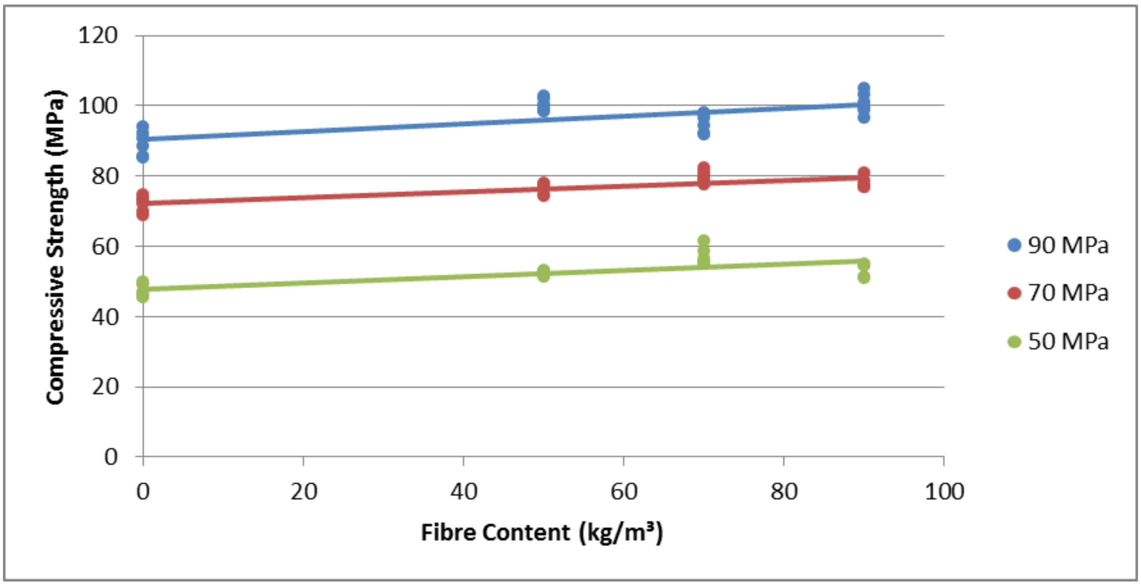

Figure 25: Compressive Strength and Fibre Content

4.3 Tensile Strength

Splitting cylinder tests were conducted to measure the tensile strength of concrete. The tensile strength results shown in Figure 26 are calculated from the principal crack strength. It is well-known that concrete is much stronger in compression than in tension. The tensile strength of concrete measured by the splitting cylinder test has been found to be in the region of 8 to 11% of the compressive cube strength of the concrete (Illston & Domone, 2008).

The results in Figure 26 show a clear increase in tensile strength with increasing concrete compressive strength. The results also show that fibre content has little influence on principal crack tensile strength of the concrete. This result was expected because steel can only carry significant tensile load after a crack has formed in the concrete. Fibres bridging the crack transfer load across the crack. Principal crack stress is governed by concrete strength and therefore fibre content does not play a significant role. A similar result is found for the principal crack strength results. While the splitting cylinder test gives a good measurement of the concretes tensile strength it can be concluded that this test is not a good measure of the fibre behaviour in the concrete.

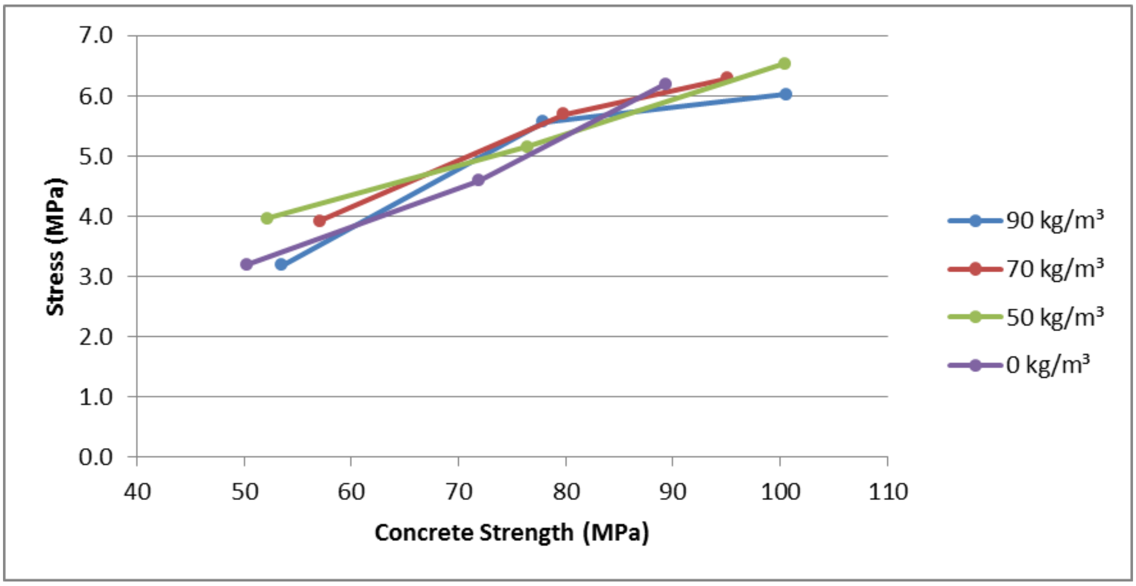

Figure 26: Splitting Cylinder Principal Crack Strength Results

In order to measure the effect of the fibres, the post crack behaviour of the sample must be examined. The stress strain behaviour of two different samples is shown in Figure 27. The inclusion of fibres gives the sample a significant amount of post crack load carrying capacity. Both samples cracked at a tensile stress of approximately 6.5 MPa. While the sample with no fibres fails in a brittle manner, the sample with fibres continues to carry load. It should be noted that this graph only shows the initial section of the stress strain relationship and the sample containing fibres continues to carry load up to high strains.

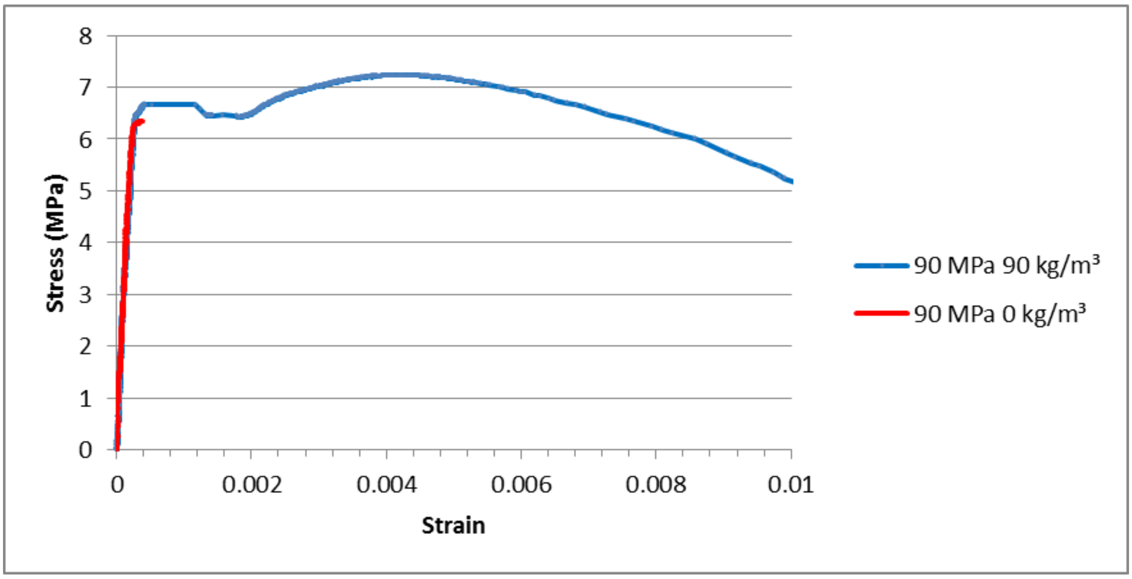

Figure 27: Stress vs Strain Plot Demonstrating the Effect of Fibres on the Tensile Properties of Concrete

Quantifying post crack strength can be done by various methods. The first comparison made here is to use the maximum stress results of the splitting cylinder test with the results shown in Figure 28. The influence of the fibres on the tensile strength is now more pronounced. Similar to the principal crack strength results shown Figure 26, there is an increase in tensile strength with increasing compressive strength; however, higher stresses are now measured for samples with higher fibre contents.

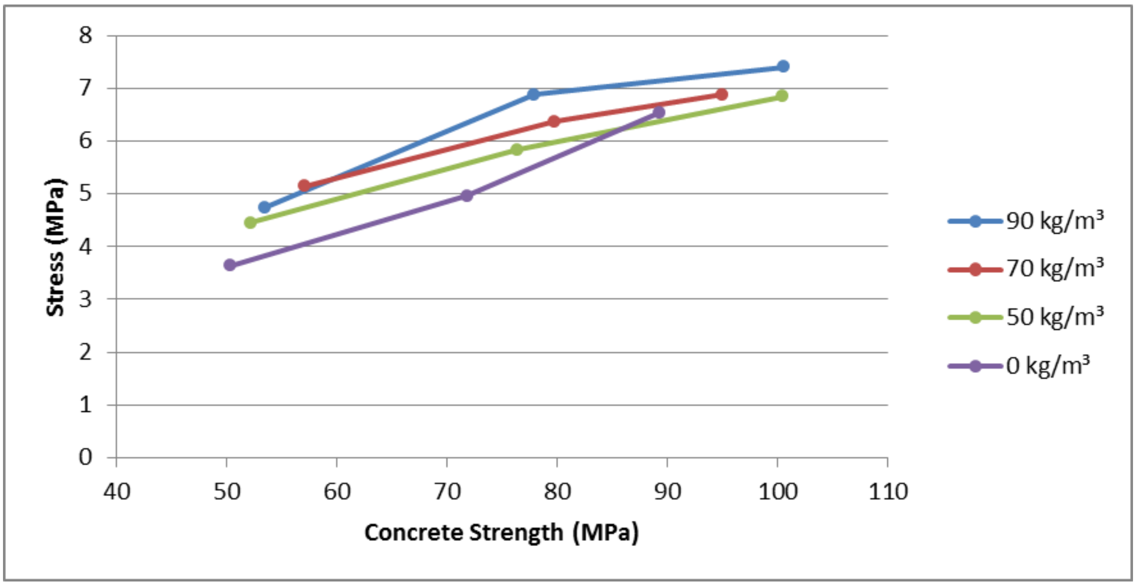

Figure 28: Splitting Cylinder Maximum Strength Results

Another method of evaluating post crack performance is made by evaluating the strain energy. Strain energy must be calculated up to a specific deformation. For the purposes of comparison, deformation limits of 0.05 mm, 0.25 mm, and 1 mm crack opening were selected. These results are shown in Figure 29 to Figure 32. It is clear from these graphs that at any strain level the presence of any quantity of fibres vastly improves the strain energy performance. At low strain levels there is little difference in the strain energy for different fibre contents; however, as the strain increases the benefit of increased fibre content becomes apparent. There is an increase of strain energy with increased concrete strength. As reported by Abu-Lebdeh et al. (2011) this result can be expected as the increased bond between the fibres and concrete matrix increases the energy required to mobilise and pull out the fibres. For lower strains the effect of the concrete strength on strain energy is greater than that of fibre content. At higher strains this effect becomes less pronounced, to the point where increasing concrete strength no longer results in increased strain energy. It is at this point that increased fibre content starts to have more influence on strain energy than the concrete strength.

High yield steel yields at a strain of approximately 0.001957 (Robberts and Marshall, 2009). In Figure 27 we saw that the maximum load was reached at a strain of approximately 0.0042. Therefore the fibres reach their maximum load caring capacity at a strain greater than that of the steel mesh. When exposed to the same strain the mesh will yield before the ultimate load caring capacity of the steel fibres is reached. Because the loading of a UTCRCP is flexural in nature placing the mesh in the centre of the layer and letting steel fibres be exposed to the greater strain capitalises on the advantages of both materials. The strain values presented in Figure 31 and Figure 32 represent large strains that would result in large deflections. In reality this level of deflection may never be reached in a road pavement due to serviceability requirements such as riding quality and allowable rut depth.

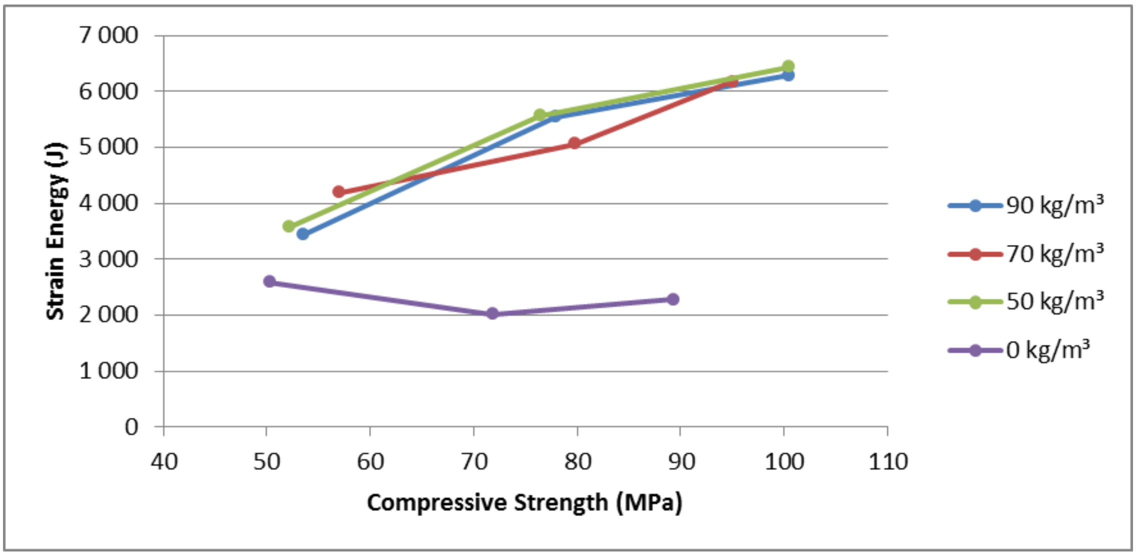

Figure 29: Strain Energy at 0.001 Strain (0.05 mm Deflection)

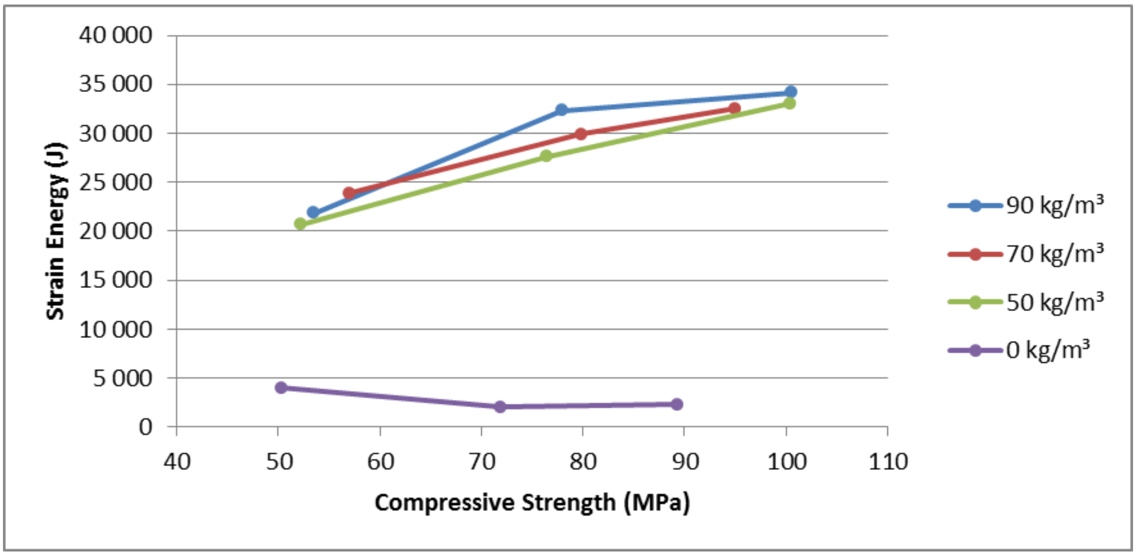

Figure 30: Strain Energy at 0.005 Strain (0.25 mm Deflection)

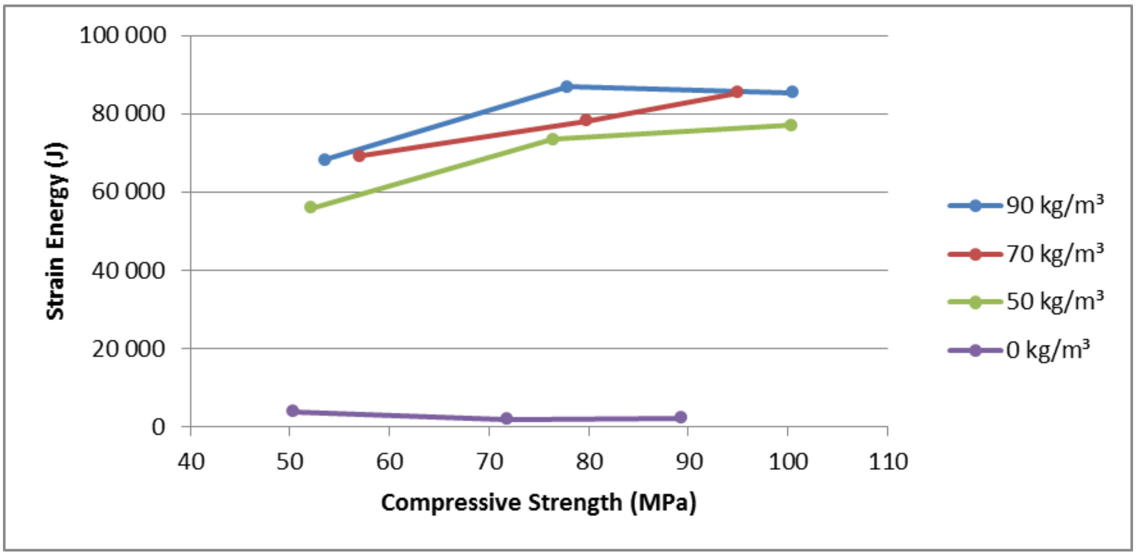

Figure 31: Strain Energy at 0.015 Strain (0.75 mm Deflection)

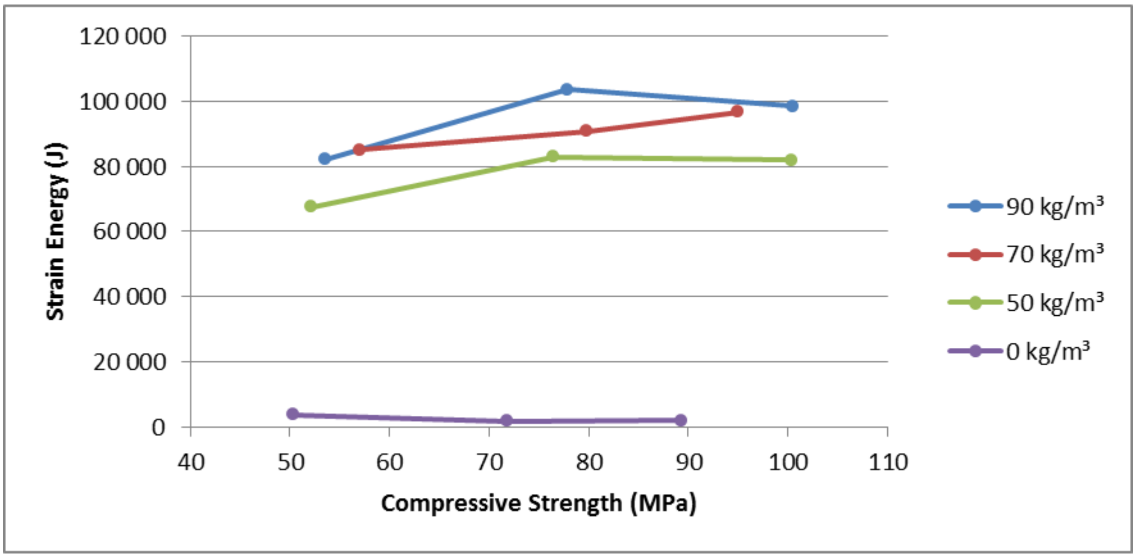

Figure 32: Strain Energy at 0.02 Strain (1 mm Deflection)

These results indicate that in terms of tensile behaviour there is a complex interaction between the fibre content and the concrete strength. Better performance is not necessarily derived by increasing the fibre content and concrete strength. The application of the material should also be taken into consideration, particularly the magnitude of the expected or acceptable tensile strains. For small strains, pre and post crack strength and energy absorption is governed principally by the concrete strength. Where wider cracks are anticipated or allowable, fibre content becomes the prevailing factor. However, it is also necessary to consider the concrete strength when selecting fibre content - increasing the concrete strength above a certain level will not lead to an increase in performance.

Increasing the concrete strength increases the cost of the concrete. Additionally increased fibre content also results in a higher material cost; therefore, there are economic consequences in selecting specifying the concrete mix design. Hence a good understanding of the situation and desired performance characteristics is vital for the efficient and correct application of fibre reinforced concrete.

Where the tensile performance is important the splitting cylinder test can be used to quantify the performance for the purposes of design input. It may also be useful in the concrete mix design process and as a quality control test. The presence and effect of fibres can be quantified if lateral displacement is measured as recommended by Denneman et al. (2012). Tensile performance is a complex interaction between concrete strength and fibre content that changes with increasing strain.

4.4 Young's Modulus

An important material property used as input into design is Young's Modulus. Linear elastic behaviour is often assumed, and this means that stress is directly related to strain. Non-linear finite element analysis allows for the more complex material behaviour to be taken in to account. Here the true stress strain relationship of the material can be used to more accurately model the behaviour of the structure under load.

The typical stress-strain behaviour of one of the cylinder samples tested is shown in Figure 33. The data recorded for the three loading cycles is plotted. While the stress-strain relationship is close to linear, it can be seen that it is not exactly linear, particularly at higher stresses. It can also be observed that after a load cycle has been applied some permanent deformation is recorded, particularly after the larger load level has been applied.

Young's Modulus or modulus of elasticity was measured for each of the concrete batches used. The ASTM C649-02 (ASTM, 2009b) assumes linear elastic concrete behaviour and thus Young's Modulus is approximated by the Chord Modulus of elasticity. Three samples were tested per batch and the mean of the results was taken. The results of Chord Modulus are presented in Figure 34.

It is known that modulus of elasticity increases for increased compressive strength; this is confirmed by the results obtained in this experiment. While there is some difference in stiffness between samples with differing fibre contents this difference appears not to be significant.

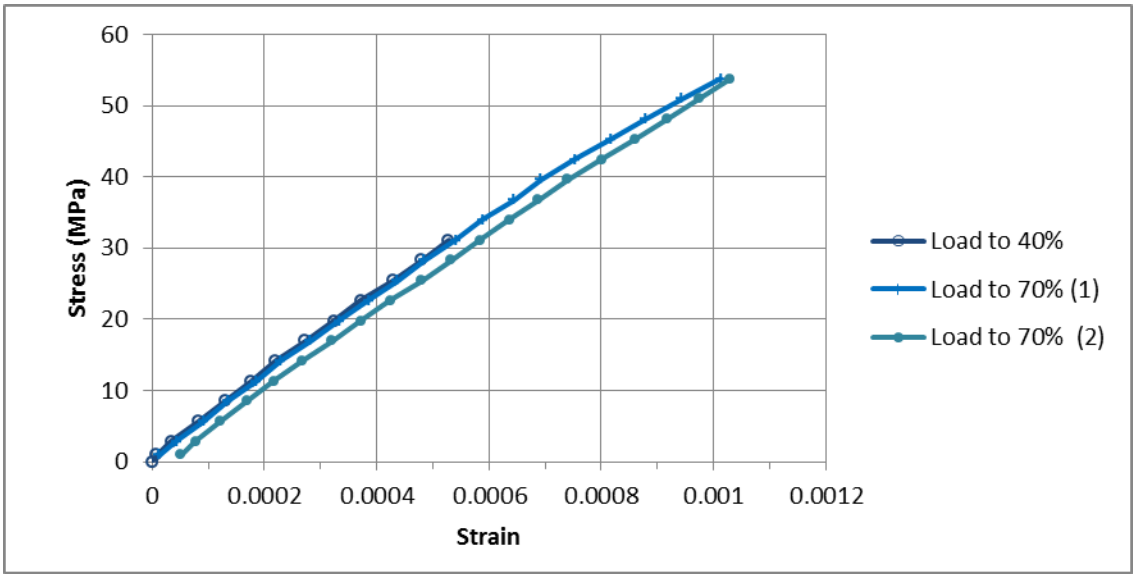

Figure 33: Typical Stress - Strain Curve from Results of E-modulus Test

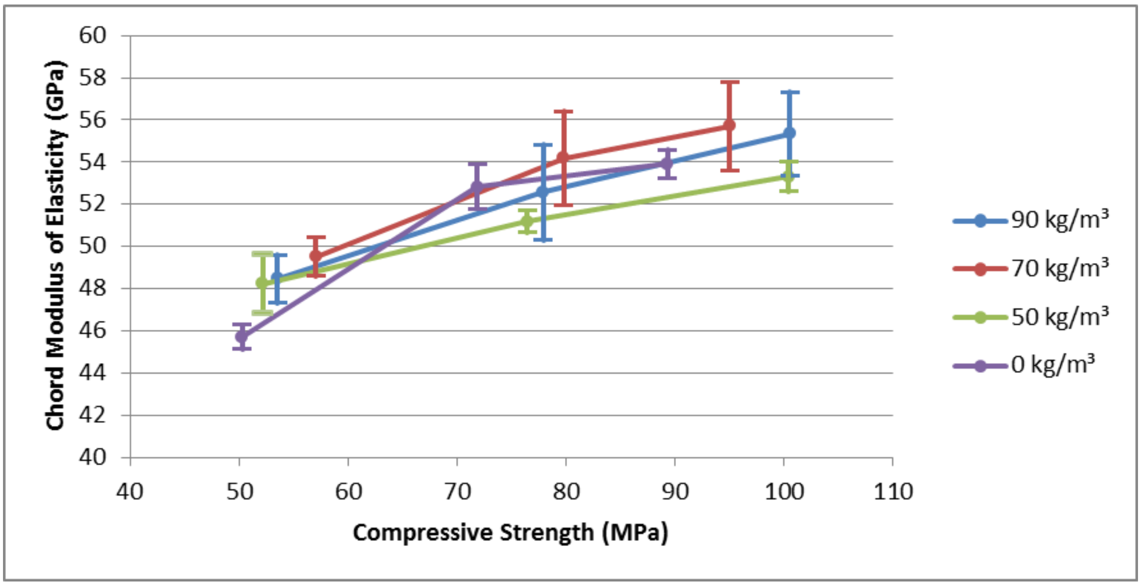

Figure 34: Chord Modulus of Elasticity for Different Fibre Contents (One Standard Deviation above and below Mean Indicated)

Concrete behaviour is often assumed to be linear elastic. During the data collection for this experiment more frequent measurements were taken during the testing, allowing a more detailed stiffness-stress relationship to be determined. The average stiffness for each batch has been plotted against the applied stress in Figure 35. The results clearly showed that the stress-strain response was non linear. A definite decrease in stiffness is recorded with increase in applied stress. Even at low levels of applied stress this behaviour is seen. The change in stiffness is significant and a decrease of as much as 20 GPa is possible. Concrete strength would appear to be the variable with the most influence. Higher concrete strength results in initial higher stiffness and a more gradual decrease in stiffness. Weaker concrete shows a more rapid decrease in stiffness with increase in applied stress.

While fibre content does not appear to play a large role in influencing the stiffness, fibre content appears to have a much greater influence on stiffness in the lower strength concrete.

When the stiffness stress relationship is normalised both in terms of initial stiffness and ultimate stress a very consistent relationship is seen. This is plotted in Figure 36. Initial stiffness is determined as the chord modulus from 0.0005 strain to stiffness at 10% of the ultimate compression stress. A polynomial function was fitted to this data, the equation is given on Figure 36. Using this relationship more detailed finite element analysis of fibre reinforced concrete is possible.

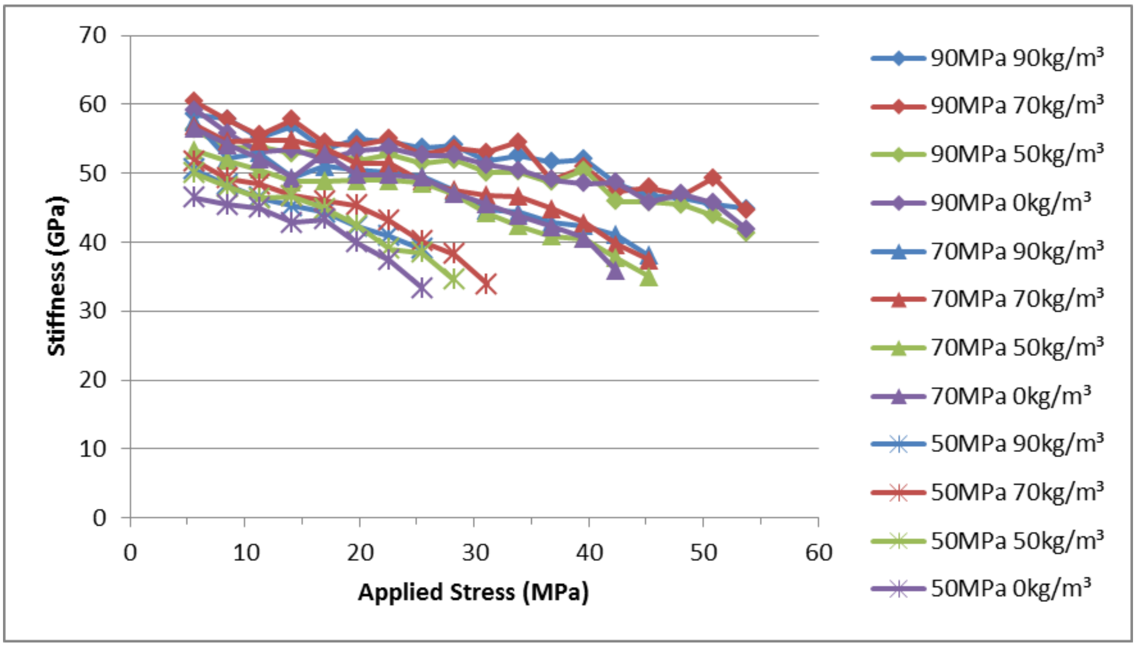

Figure 35: Stiffness as a Function of Applied Stress

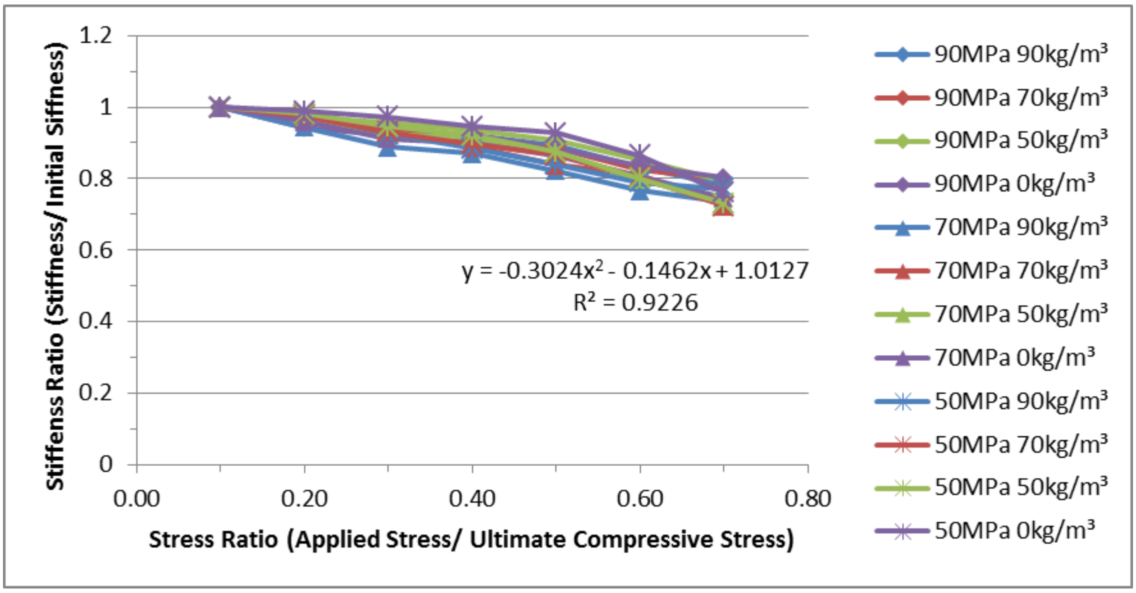

Figure 36: Normalised Stress and Normalised Strain Relationship

From these results it can be seen that the stress strain relationship of concrete is non-linear. While the assumption of linear elastic may suffice for simple design calculations, it is a simplification. The use of more advanced finite element packages makes it possible to input the nonlinear behaviour. Non-linear behaviour should be used when doing FEM.

Young's Modulus tests are not normally conducted as part of normal construction quality control tests during road construction. This test does, however, provide useful information for the purposes of modelling the pavement. It is recommended that Young's Modulus be measured and taken into account during the pavement design and mix design phases.

4.5 Poisson's Ratio

The Poisson's ratio for concrete recommended for structural design by the SABS is a value of 0.2 (SABS0100) although it is known that it can vary between 0.15 and 0.25 (Robberts and Marshall, 2009). Poisson's ratio is an important parameter used when doing finite element modelling and other calculations. The values measured on samples of fibre reinforced concrete are shown in Figure 37. The Poisson values tested ranged between 0.15 and 0.21, within the anticipated range. Poisson's ratios peaked at a concrete strength of approximately 80 MPa. Poisson's ratio does not appear to be influenced by fibre content, except perhaps at lower concrete strengths.

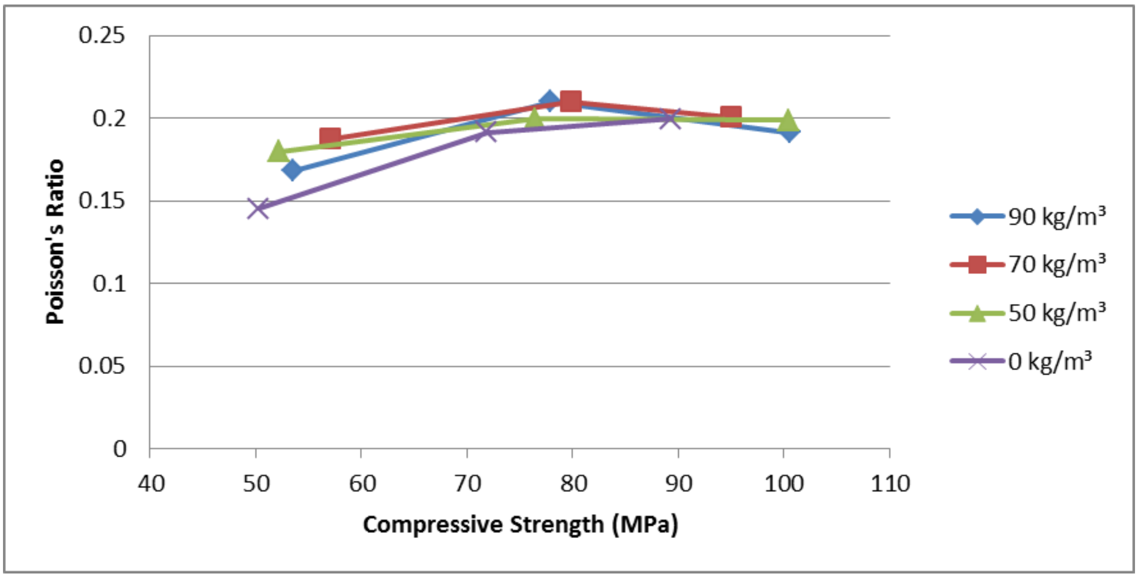

Figure 37: Poisson's Ratio

Poisson's ratio is not often used as a standard quality control test for concrete pavements. It may, however, be worth taking into consideration the actual Poisson's ratio when conducting finite element modelling of the pavement.

4.6 Slabs

The advantage of the slab test is that its thickness is the same as that of the constructed pavement. This overcomes the size effect giving a better indication of the actual pavement performance. Additionally, the loading replicates bending that will be similar to the forces that the constructed pavement would have to resist. Slab tests do not only test the material properties of the concrete, but also the strength of the whole system, as the contribution of the reinforcing bars is also measured.

Many different parameters can be measured from a slab test. These can often give conflicting results as to the best performing concrete mixture. It is therefore necessary to understand how the result is determined and its relevance to the characteristic that is of most importance to the ultimate purpose of the concrete pavement. In this section, the results of the slab testing are presented and discussed.

4.6.1 Modulus of Rupture

At the onset of loading the relationship between load and deflection is linear and this linear relationship, which continues up to the point where the principal crack occurs, is largely a measure of the tensile strength of the concrete. Without steel reinforcing or steel fibres, this would be the ultimate load carried by the sample. The stress in the outer fibres of the concrete, when the failure occurs, is known as the MOR. Thus the tensile strength of the concrete can be determined from the slab test. The results of the slab MOR are shown in Figure 38. As expected, MOR is largely influenced by concrete strength while fibre content has a marginal effect. The tensile strength of the concrete relative to its compressive strength can also be seen in this figure. Tensile strength of concrete can be expected to be approximately 10% of the compressive strength. The results shown fall within this range.

Splitting cylinder tests were also used to measure the tensile strength of concrete. A comparison of the tensile strength measured by splitting cylinder and the slab MOR is given in Figure 39. The splitting cylinder test strength results underestimate the tensile strength achieved by the slabs. The implication of this is that if splitting cylinder tests are used to quantify the tensile strength of UTCRCP an adjustment factor should be used in order to get a realistic tensile strength of the constructed UTCRCP.

The good correlation between MOR measured from slab tests and the tensile strengths measured from the splitting cylinder test is an indication that only one of these tests is needed for quality control purposes. The advantage of using the Slab MOR is that it is calculated from the same test as the energy absorption. Energy absorption to certain deflection limits is currently used as a construction quality control. Omitting splitting cylinder testing and performing slab tests as a quality control would reduce the time, energy, and cost of on-site quality control testing during the construction of UTCRCP.

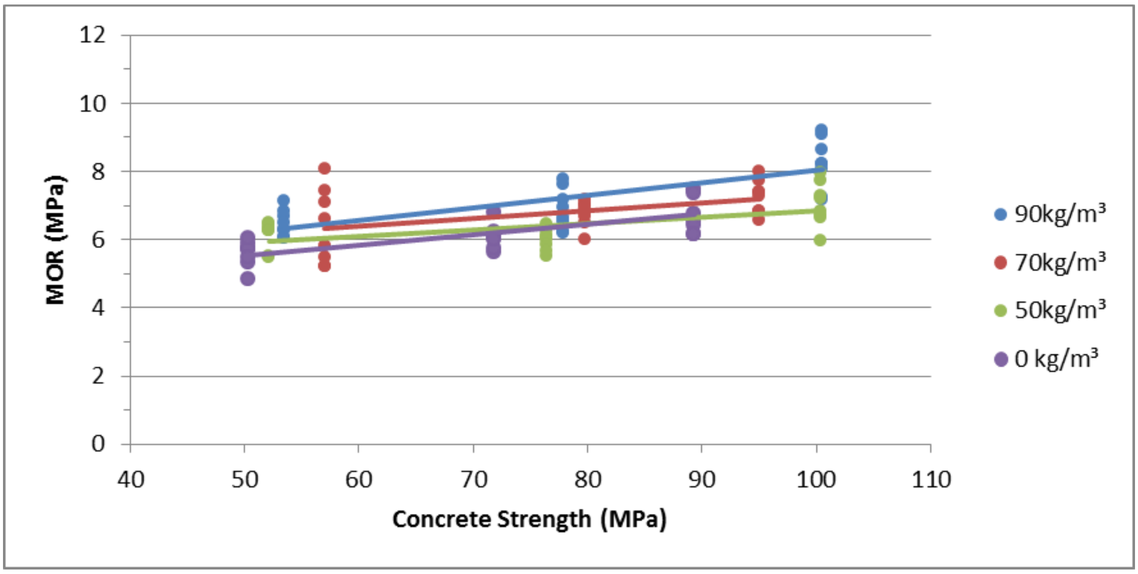

Figure 38: MOR of Slabs

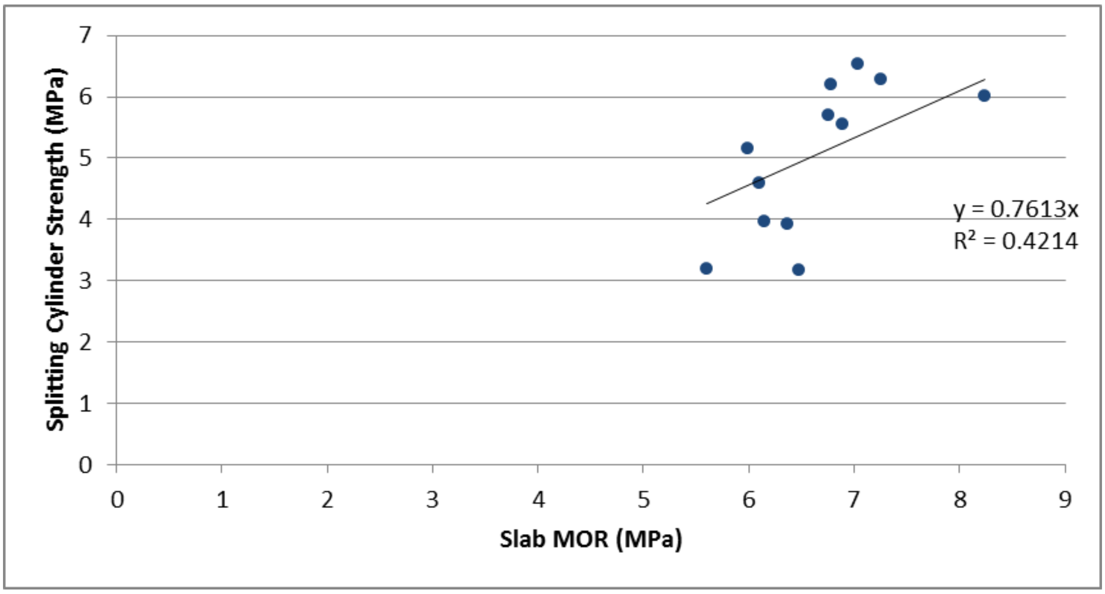

Figure 39: Comparison between Splitting Cylinder Strength and Slab MOR

Current pavement design theory and observations of UTCRCP has indicated that an UTCRCP layer experiences high levels of stress from environmental loads. Therefore, it can be assumed that shortly after construction, the concrete material could be in a cracked state. The design philosophy of UTCRCP does not rely on the tensile strength of the concrete for mechanical performance. As the pavement has no joints, expansion of the pavement due to environmental effect must be taken up by closing cracks that have opened up. The ideal is that the pavement will contain many well-spaced micro cracks that will allow for expansion and contraction but will limit the ingress of water (Perrie & Rossmann, 2009). The operational philosophy of the pavement requires that the concrete be cracked. (de Larrad, 2005). Measuring the tensile strength of the concrete therefore may be viewed as a fruitless exercise. Alternatively a low tensile strength may be desirable as it is required that the concrete cracks.

Understanding the thermal expansion characteristics of UTCRCP, as well as the shrinkage and creep behaviour, would be required before the desired tensile strength of the concrete can be selected.

4.6.2 Maximum Load

Maximum load gives an indication of the overall strength of the concrete, reinforcing steel and steel fibres as a system. However, in reality, when the UTCRCP is constructed on its foundation layers this measurement has little relevance as the slab will be continuously supported and would not be able to reach the deflections at which the maximum load is reached.

The maximum load carried by the slabs is given in Figure 40. Based on the results displayed there would appear to be little benefit in increased concrete strength. Fibre content has some effect on the maximum load carried but its effect is not large. A large contribution to the bending resistance is the reinforcing bars; the number of bars was constant for all the slabs. At high fibre contents the steel fibres did make some contribution to the reinforcing.

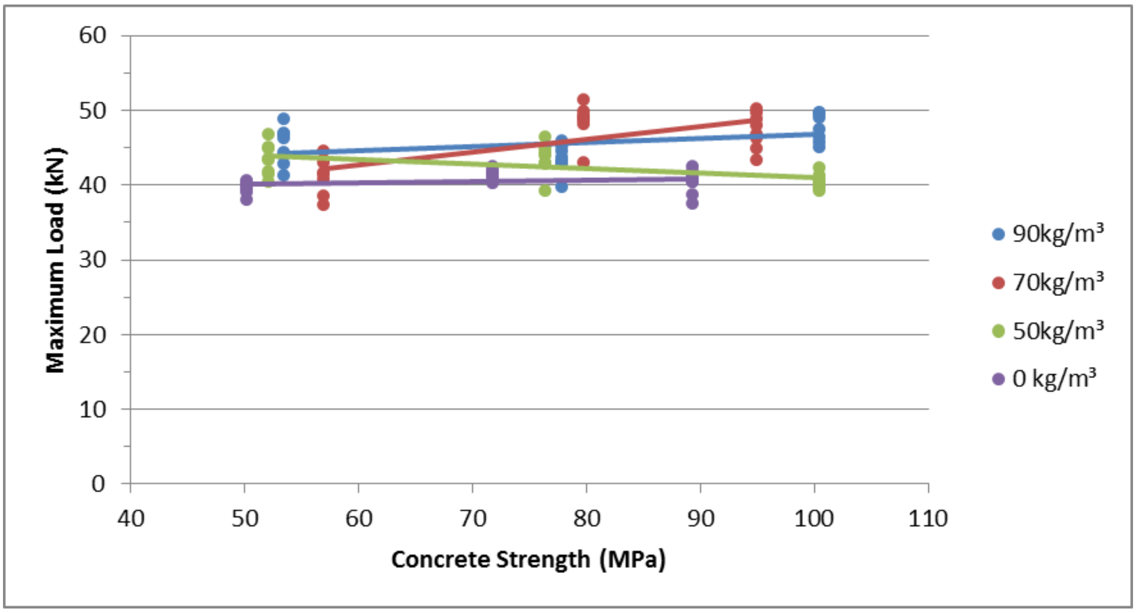

Figure 40: Slabs Maximum Load

Subsequent to pre-cracking and the application of cyclic loading the slabs were tested to failure. Thus the change in maximum load can be compared. These results are displayed in Figure 41. It was found that in all cases little change in the maximum load carrying capacity occurred after pre-cracking. Additionally, cyclic loading up to a million cycles also had minimal effect on the maximum load carrying capacity of the slabs.

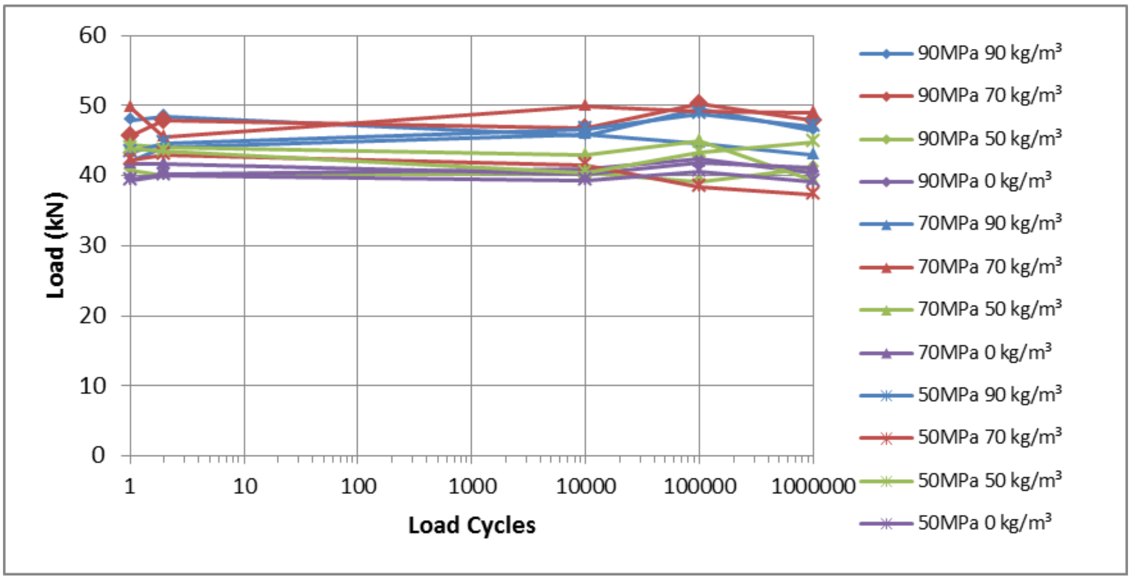

Figure 41: Maximum load after cyclic loading

While testing for the maximum load is an important parameter for structural concrete in suspended slabs or beams, its relevance to the performance of concrete for a pavement could be queried. Maximum load is carried at large deflections, in excess of any rut depth that would be tolerated on a road pavement surface. It would be difficult to justify the selection of a concrete mix based on the results of maximum load. The insensitivity to concrete mix demonstrated by these results also demonstrates that maximum load would make a poor construction quality control measure for UTCRCP.

4.6.3 Energy Absorption- Without Cyclic Loading

Energy absorbed is a measure of the work done in order to deflect the sample to a certain point. Energy absorption is determined from the results of a monotonic load test. This measure is preferred when testing fibre reinforced concrete as it gives a better measure of strength with more differentiation between samples than a parameter like maximum load or MOR. Energy absorption is calculated up to a specified deflection. Energy absorption is one of the quality control tests required for UTCRCP. At a deflection of 25 mm a minimum energy of 700 J must be reached for the concrete mixture to be accepted (COLTO, 2010). The energy absorption requirement specified in COLTO is for tests performed on a disk sample so it cannot be directly applied to the results of this experiment. However, the load deflection behaviour of fibre reinforced concrete, whether tested by disk or slab, is similar. The assumption made here is that the best performing concrete mix tested on a slab will be the best performing mix on a disc sample.

Similar to the results of maximum load, the effect of variation of slab thickness in energy absorption calculation was not taken into account. The potential influence of variation in slab thickness was examined but was found not to affect these results.

Energy absorption is measured in Joules and is calculated as the area under the load deflection graph up to a specified deflection limit. Thus the energy absorption graph increases with increasing deflection. The results of energy absorption for un-cracked slabs is presented in Figure 42. This figure gives an indication of the energy absorption as deflection increases. After 15 mm most samples have reached their peak and there is little energy absorption beyond this point. This means that after a deflection of 15 mm the sample has broken and has very little strength left. By comparing the results with previous testing done it was found that samples tested over a greater span, the plateau is reached at a greater deflection. This could be attributed to the strain in the steel. On a shorter span there must be more rotation for the same amount of deflection. Assuming that the concrete does not crush, the crack in the sample will hinge at the top of the concrete. This means that the steel reinforcing and steel fibres will experience more strain per unit increase of deflection on a smaller span. The pull-out of the fibres and breaking of the steel is affected by this strain. It can thus be concluded that there is an inverse relationship between span distance and the deflection at which the energy absorbed no longer increases.

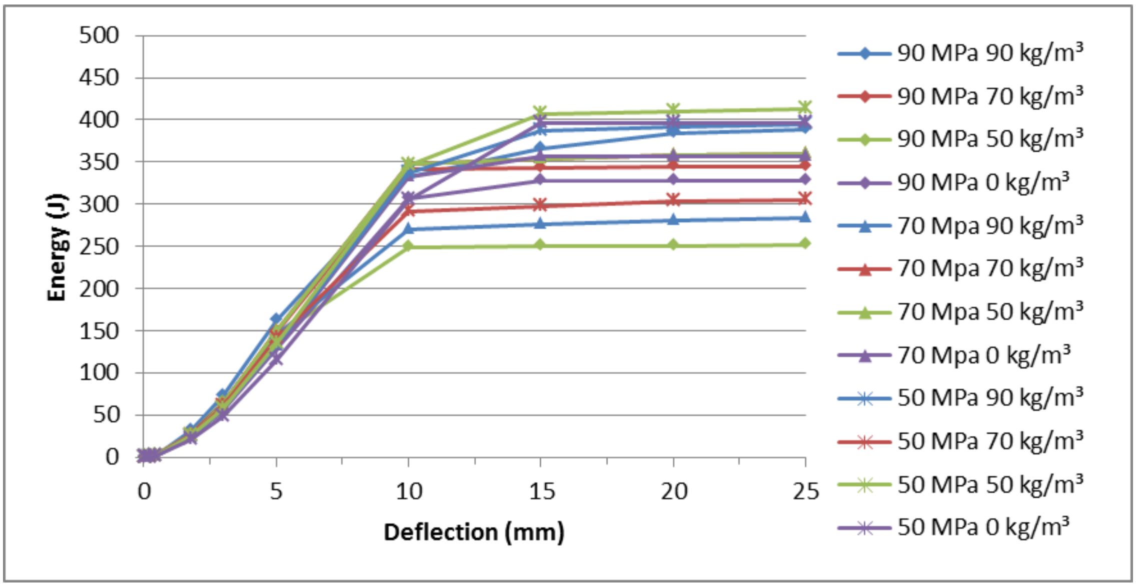

Figure 42: Energy Absorption with Deflection – Un-cracked slabs

In order to evaluate the difference in energy absorption at low and high deflections, a number of discrete deflections were chosen. A value of 1.8 mm was selected to represent a low deflection. This value was determined from the structural design code for serviceability where sag should not exceed the ratio of the span distance over 250. Mid-range energy was determined at a deflection of 10 mm and energy for large deflections was determined at 15 mm.

The energy absorption at these three deflections is plotted in Figure 43[3]. At the low deflections the energy absorbed increases with increasing concrete strength. Increased fibre content also contributes to increased energy absorption (see Figure 43a). At high deflections this trend is reversed. Energy absorption decreases with increased concrete strength and fibre content (see Figure 43c). At mid-range deflections it can be seen that the transition between these two trends, at 10 mm deflection the energy absorption is approximately constant with increasing concrete strength (see Figure 43b).

The reason for this behaviour i.e. the weaker concrete recording higher energy absorption at large deflections could possibly be attributed to the concrete crushing in compression. The crushing of the compression concrete in the slab delays the failure of the reinforcing mesh as the strain is relieved. Thus, at larger deflections more energy can be absorbed. The mode of failure could be clearly observed during testing. Figure 44 shows a slab of 90 MPa concrete being loaded at high deflection. Cracking has advanced completely through the slab. The compression block at the top of the slab is very small. Figure 45 shows a sample of 50 MPa concrete where cracking has not advanced through the slab. In the compression zone a horizontal crack is visible. This is an indication of the concrete crushing in compression.

[3] For further comparison the results for 5 and 25mm deflection are given in the Appendix A Figure A-1 and Figure A-2

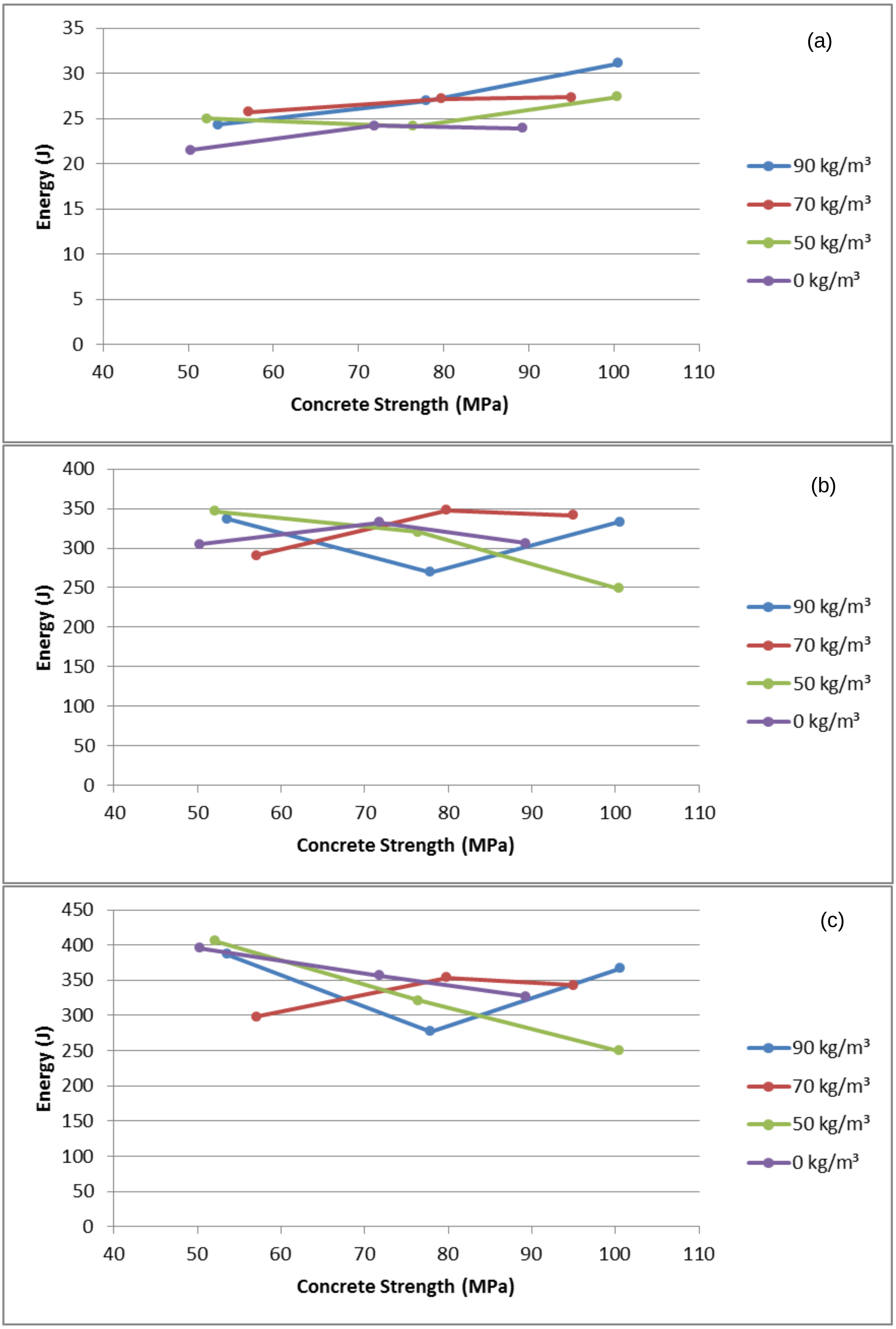

Figure 43: Energy Absorption of Un-Cracked Sample at Deflections: a) 1.8 mm, b) 10 mm, c) 15 mm

Figure 44: Slab of Batch B10- 90 MPa Concrete with Fibre Content of 70 kg/m³

Figure 45: Slab of Batch B12 - 50 MPa Concrete, No Steel Fibres

An optimal design for UTCRCP will seek to maximise the energy absorption of the pavement. Here the deflection range that best represents the long term behaviour of the pavement must be chosen. This must also be considered when using energy absorption as a quality control measure. The COLTO UTCRCP quality control specification requires minimum 700 J of absorption measured at 25 mm (COLTO, 2010). At high deflections concrete strength can be decreased in order to meet the energy specification. Using this high deflection energy absorption as a quality control target may inadvertently reward the production of weaker concrete. As road pavements experience low deflections during their service life it may be argued that low deflection energy absorption is a more valid measurement.

This proves that energy absorption values recorded at relatively large deflections could be misleading and that a low deflection energy test should be curtailed at a maximum deflection of less than 10 mm.

4.6.4 Energy Absorption- With Cyclic Loading

After the application of cyclic loading samples were loaded monotonically. The energy absorption was then determined. Energy absorptions for the same deflections as above are presented in Figure 46, Figure 47 and Figure 48[4] . There is not a marked decrease in energy absorption at low deflections between un-cracked and cracked samples. At large deflections, however, this decrease is seen.

With increased load cycles there is little decrease in energy absorption measured. From this it can be concluded that energy absorption does not provide a sensitive measure of fatigue damage. At a greater number of load repetitions, when fatigue damage is more significant, it may become a useful measure: although as indicated by Rilem, (1984) this method of measuring fatigue damage may also not be successful. It has been noted that concrete samples that are tested under monotonic loading, after cyclic loading, can have a higher strength than those which have not been subjected to cyclic loading. Reasons for this include release of residual stress under the repeated loads, deflection hardening, and increase in strength as a result of loss of capillary moisture during the course of loading (Rilem, 1984).

[4] For further comparison the results for 5 and 25 mm deflection are given in the Appendix B Figure B-1 and B-2

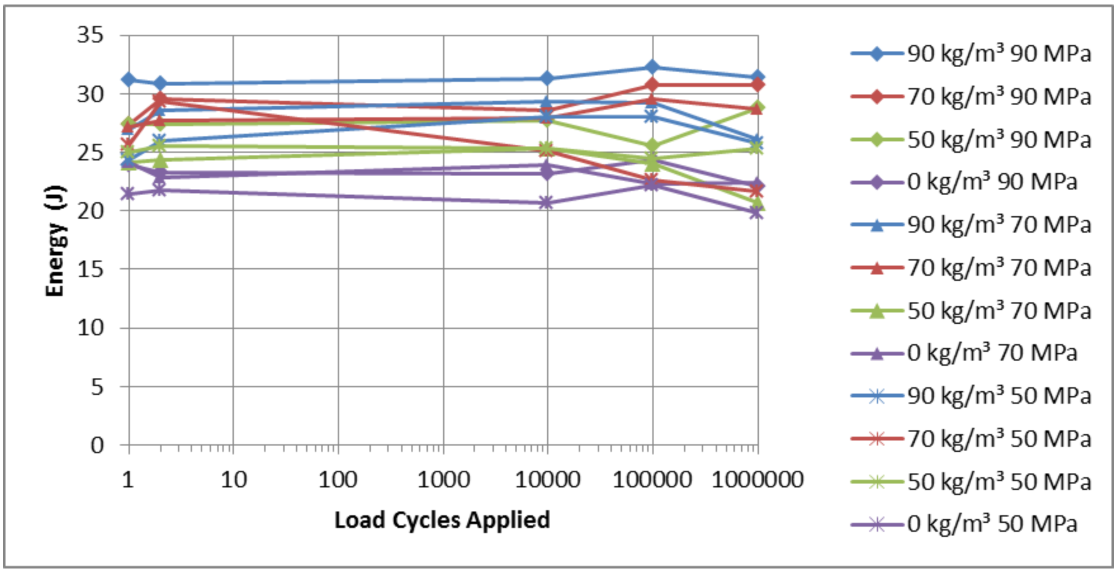

Figure 46: Energy Absorption to 1.8 mm Deflection with Cyclic Loading

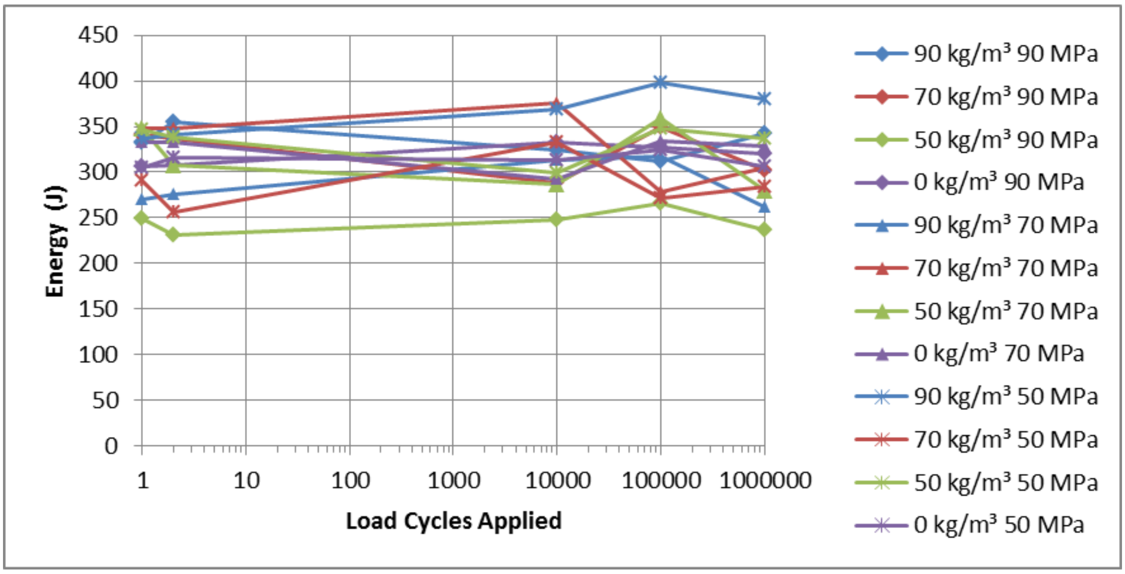

Figure 47: Energy Absorption to 10mm deflection with cyclic loading

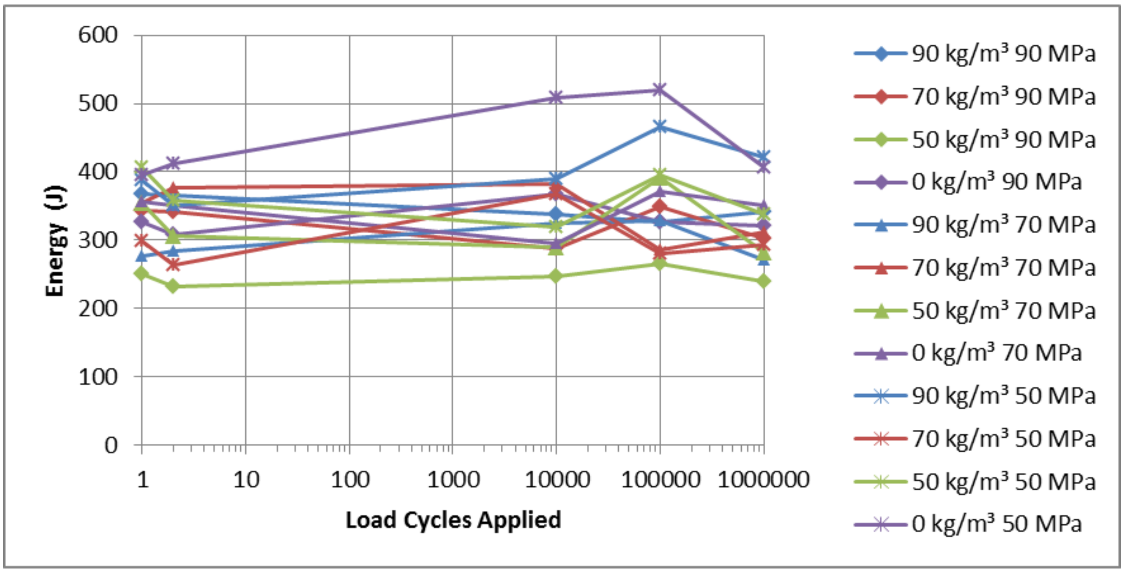

Figure 48: Energy Absorption to 15mm Deflection with Cyclic Loading

The energy absorption relationship to concrete strength for slabs subjected to 1 million load cycles is shown in Figure 49[5]. The results show similar energy absorption to static tests with no cyclic loading applied (See Figure 43) At large deflection values energy absorption is inversely proportional to concrete strength. The same comments and explanations as made for the slabs tested monotonically with no cyclic loading applied could be made here. There is little difference between the energy absorption of samples tested with and without the application of the cyclic load. It can therefore be concluded that comparing monotonically loaded slab results before and after application of cyclic loads is not an effective way of measuring fatigue damage or fatigue performance.

Energy absorption under monotonic loads may be an indicator of the strength of the material. However the suitability of this measurement as a measure or predictor of fatigue or actual pavement performance is questionable. It is also clear from these results that the deflection limit at which the energy absorption is determined must be carefully selected. It would appear that energy absorption at low deflections (less than 10 mm) is a more suitable measure for pavement performance. More research would be required to understand this relationship better.

As a quality control test Energy absorption is a straight forward method. The test is quick and relatively straight-forward to run and the results are not difficult to calculate. The disadvantage is that the test requires equipment that is not currently used for quality control tests.

As a measure of fatigue damage energy absorption under monotonically increasing loads did not produce conclusive results. At higher cycle fatigue however better results may be obtained.

[5] For further comparison the results for 5 and 25 mm deflection are given in the Appendix C Figure C-1 and Figure C-2

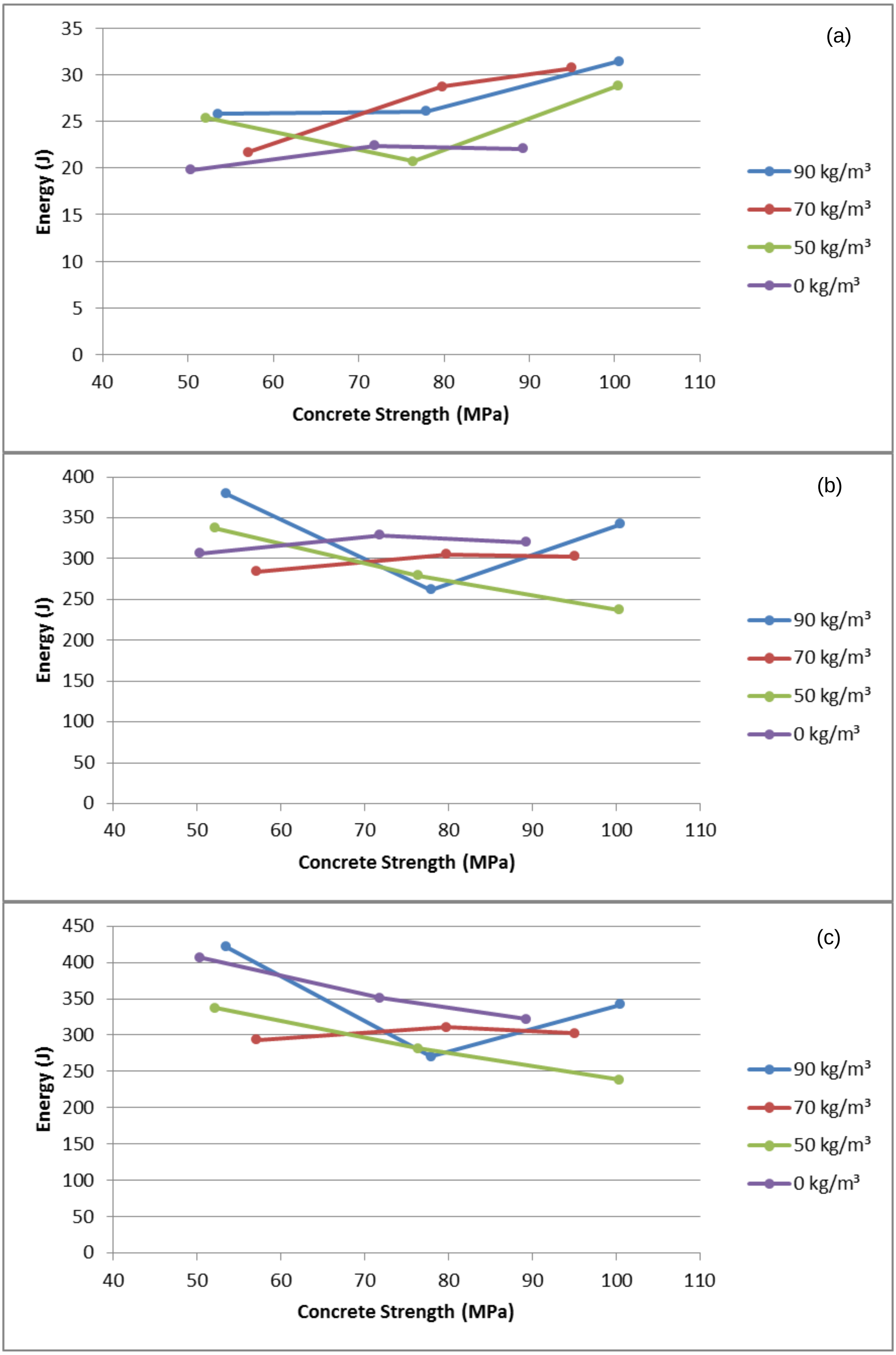

Figure 49: Energy Absorption of Samples Tested Monotonically After 1 000 000 Load Applications at Deflections: a) 1.8mm, b)10mm, c) 15mm

4.6.5 Slab Stiffness

Up to this point the results for the static load tests have been presented. While these quantify material properties such as maximum load, tensile strength or compressive strength, the results shed little light on which concrete mix would make for the longest lasting UTCRCP. In the next four sections the results of the fatigue performance parameters will be presented.

As described before, the stiffness or slope to first crack gives an indication of the flexural behaviour at low deflections. The influence of concrete strength and fibre content as well as the uncracked, cracked and fatigue effect will be inspected.

Slope to first crack gives a measure of the stiffness of the slab in its linear elastic range. It is calculated from the results of the monotonic load tests. Slabs were tested in an un-cracked state, a cracked state, and after application of cyclic load. It can be expected that as the slab weakens a decrease in the stiffness will occur and consequently the slope to principal crack will reduce. It can be expected that a large decrease in stiffness should be observed between the un-cracked and cracked state. Further damage due to the application of fatigue loading should also result in a decrease in stiffness.

A detailed comparison of the stiffness at different load cycles are plotted against concrete strength in Figure 50. Un-cracked stiffness increases with concrete strength and there is little differentiation between fibre contents. This is in line with the expected behaviour; stiffness can be directly related to the Young's modulus of the concrete. Because the section is un-cracked, the reinforcing plays little role in carrying load.

A large reduction in stiffness is seen after samples have cracked. For cracked sections increasing fibre content and increased concrete strength results in increased stiffness. Fibre content has a significant influence on the stiffness. From the results it can be observed that fibre contents of 90 kg/m³ and 70 kg/m³ have similar performance at high concrete strengths. However at increasingly high load cycles the 70 kg/m³ samples seem to out-perform the 90 kg/m³ samples. The number of samples tested is however insufficient to confirm that there is an optimum fibre content. Further testing would be required to confirm the observed trend.

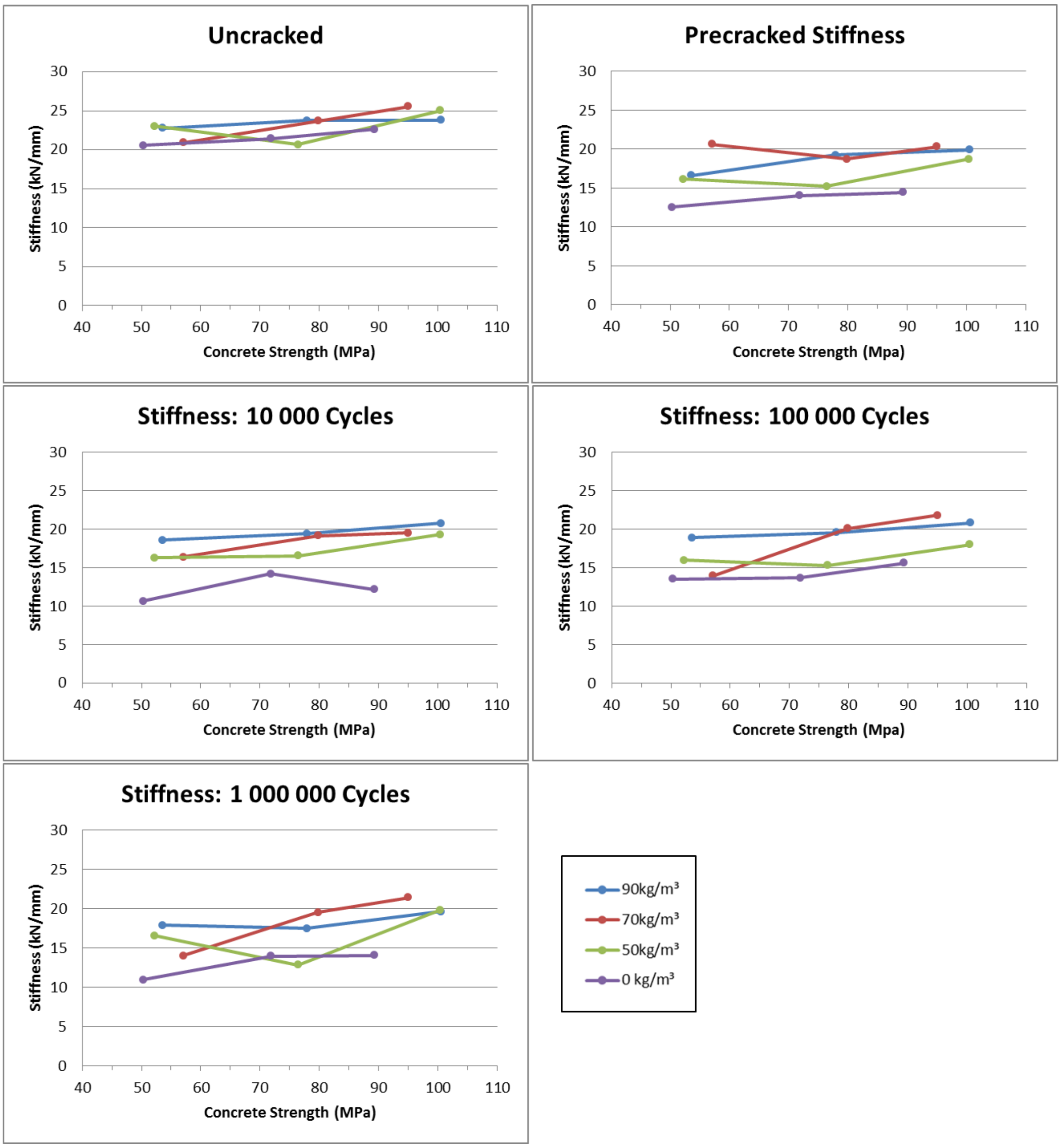

Figure 50: Stiffness Vs Concrete Strength for Each Load Cycle Level

The change in stiffness with repeated loading is displayed in Figure 51. A large reduction in stiffness is observed for all samples between the un-cracked and cracked state, Thereafter there appears to be only minimal decay with cyclic load. The increase in stiffness seen at the higher repetitions of load cycles confirms the findings of Rilem (1984). The possible causes for this phenomenon may include: release of residual stress under the repeated loads or strain hardening. It is also possible that an insufficient number of load cycles have been applied and that the samples are not yet significantly weakened.

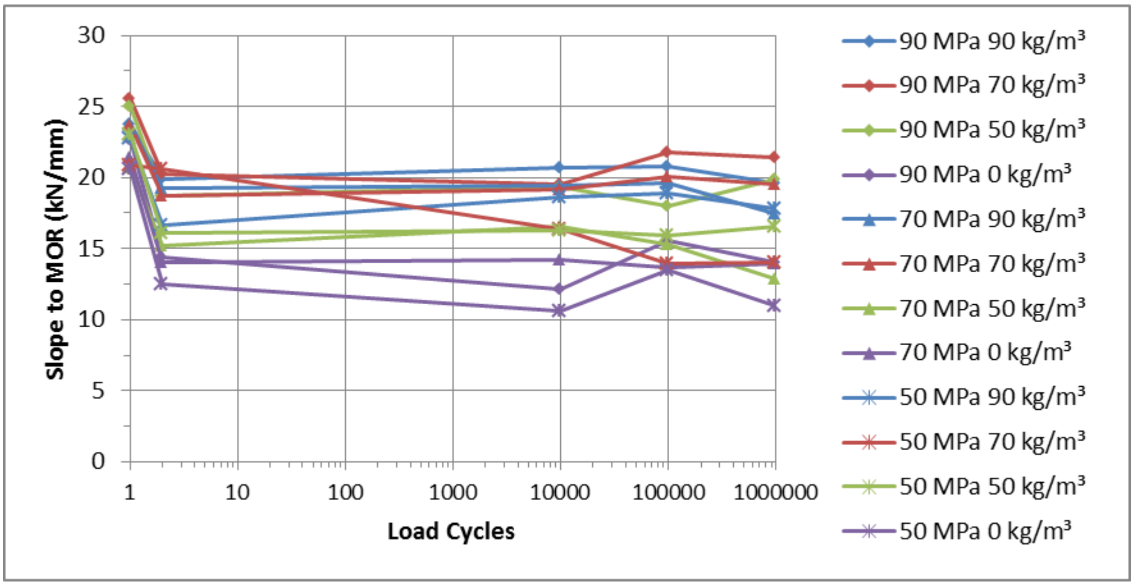

Figure 51: Stiffness with cyclic loading

4.6.6 Cyclic Loading: Stiffness

During the application of cyclic loading information was also collected. Load and deflection data was collected every 500 cycles. From this data a number of parameters could be determined. These were stiffness, hysteresis loop width and maximum and minimum deflection. The results of these are given below.

At the onset of cyclic loading the stiffness drops rapidly but then slows to a constant steady decrease in stiffness with repeated load cycles. This initial rapid drop of stiffness is seen across all concrete strengths and all fibre contents. After the first 100 000 load cycles the stiffness stabilises and decreases at a steady but gradual rate. The results of the stiffness are given in Figure 52. The same data plotted on a logarithmic scale is plotted in the Appendix D. Irrespective of the scale that the data is plotted against, it is clear that up to 1 million cycles none of the samples show a rapid decrease of stiffness that could be associated with a failure or end of life condition.

In terms of magnitude, slab stiffness is directly proportional to compressive strength. Fibre content appears to have a minor influence on stiffness (Figure 53a)[6]. In order to determine the significance of this difference one standard deviation above and below the mean is shown. Examining this, the contribution of fibre content is significant. After the application of 1 million load cycles, samples with higher fibre contents show less reduction in slab stiffness than samples with low or no fibres (Figure 53b).

[6] For further comparison the results of Figure 53 are plotted on a logarithmic scale in Appendix D Figure D-1

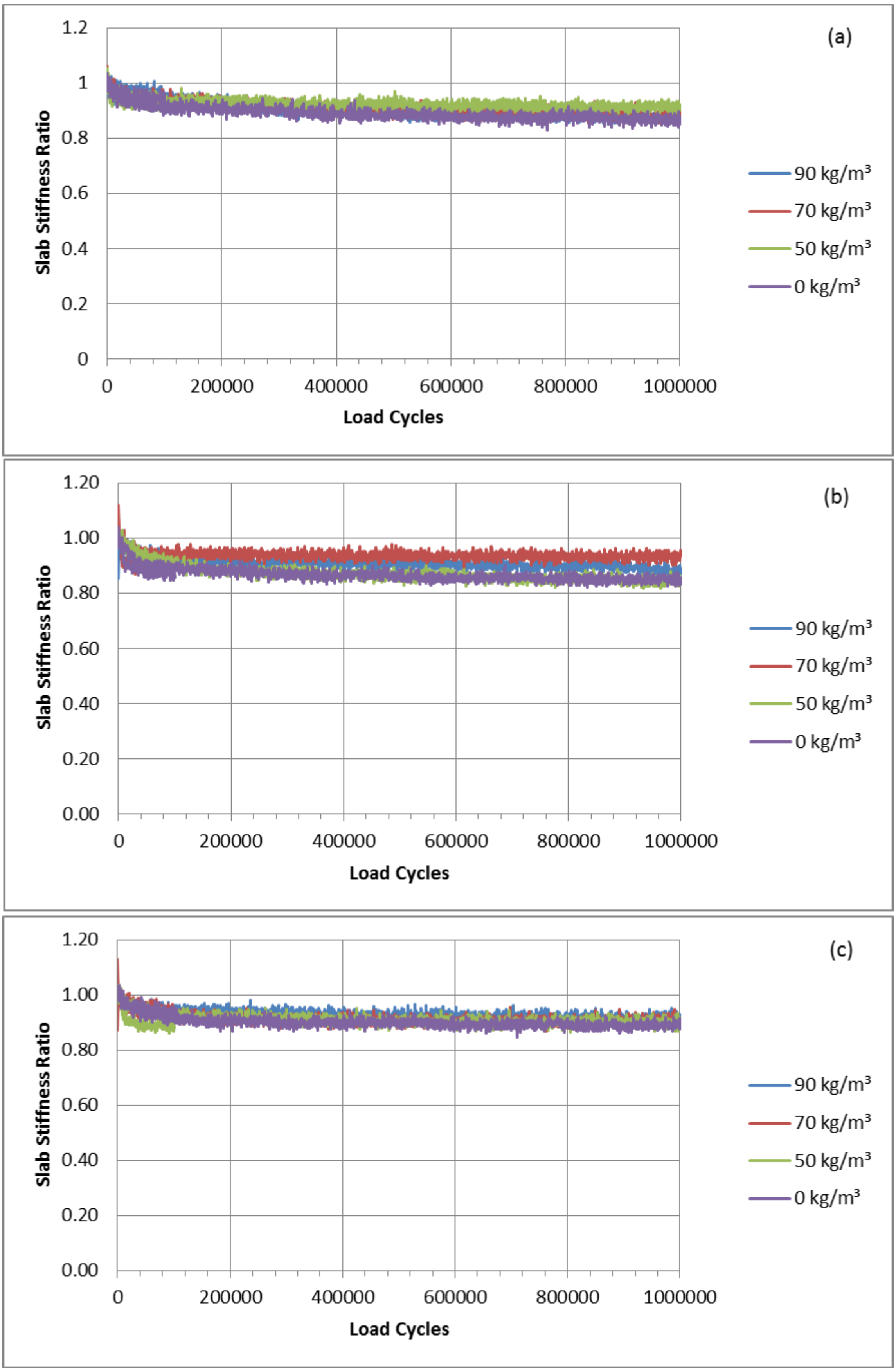

Figure 52: Slab Stiffness as Ratio of Initial Slab Stiffness for: a) 90 MPa b) 70 MPa and c) 50 MPa

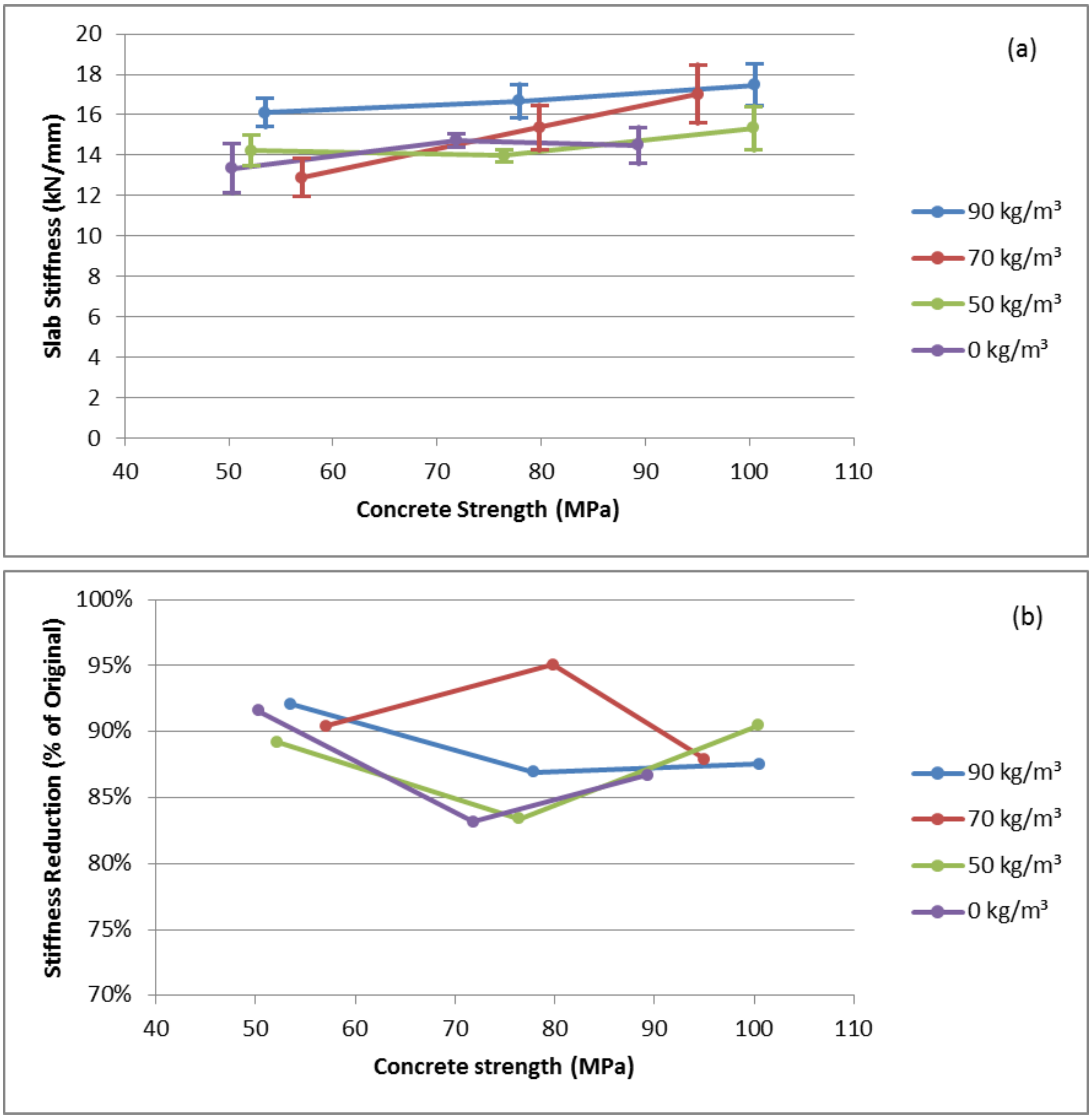

Figure 53: Slab Stiffness: a) After 500 load Cycles, b) Change as % of Initial After 1 000 000 Load Cycles

Measuring the stiffness during cyclic load application is advantageous. More regular measurements of change in stiffness of the sample can be made. Variability between samples does not affect the results. Additionally fewer samples need to be prepared. Fewer samples could be tested to greater number of load repetitions in the same amount of time, yielding more information than the monotonic load tests.

4.6.7 Cyclic Loading: Loop Width

The load deflection relationship between loading and unloading is not necessarily the same. This is called hysteresis. Hysteresis is a measure of the energy absorbed by a sample during loading and unloading. It can be used to not only evaluate the magnitude of fatigue damage but also as an early warning of fatigue failure. In this experiment loop width is used to quantify the hysteresis behaviour during the experiment. In experiments conducted by Do et al. (1993) it was found that hysteresis energy would initially decrease under application of cyclic loading but would then increase to levels higher than the initial levels as the material approached failure. It can therefore be expected that loop width will initially decrease but would then increase as the sample approached failure.

The first set of loop width measurements for each sample was found to be excessively noisy. To remedy this, the second set of recorded measurements is used as a baseline measure. The cause of the variation in the first set of measurements is believed to be as a result of the sample having not yet bedded into the loading apparatus. Thus the loop widths are normalised against the loop width after the first 500 load cycles had been applied.

The loop width results with increasing load applications are shown in Figure 54. Initially a rapid increase in loop width is seen. After this initial period the loop width growth stabilises to a slow and steady growth under further load cycle applications. This is not in line with the behaviour that was anticipated. It was expected that loop width must first decrease. This behaviour is however not seen.

Figure 55 shows the loop width results at the start and end of testing. In Figure 55 a), it can be seen that the loop width decreases with increasing concrete strength. As expected the smallest loop width is measured with the higher strength concrete and high fibre contents. There is also some distinction between high fibre and low fibre contents. High fibre content appears to result in smaller loop widths. After one million load cycles had been applied the magnitude of the loop width had increased. As a percentage of the initial loop width all samples show a similar increase in loop width. Samples at concrete strength around 70 MPa show the largest increase in loop width as compared to higher and lower strength concrete. The sample of 70 MPa concrete and a fibre content of 70 kg/m^3 measured the lowest increase in loop width.

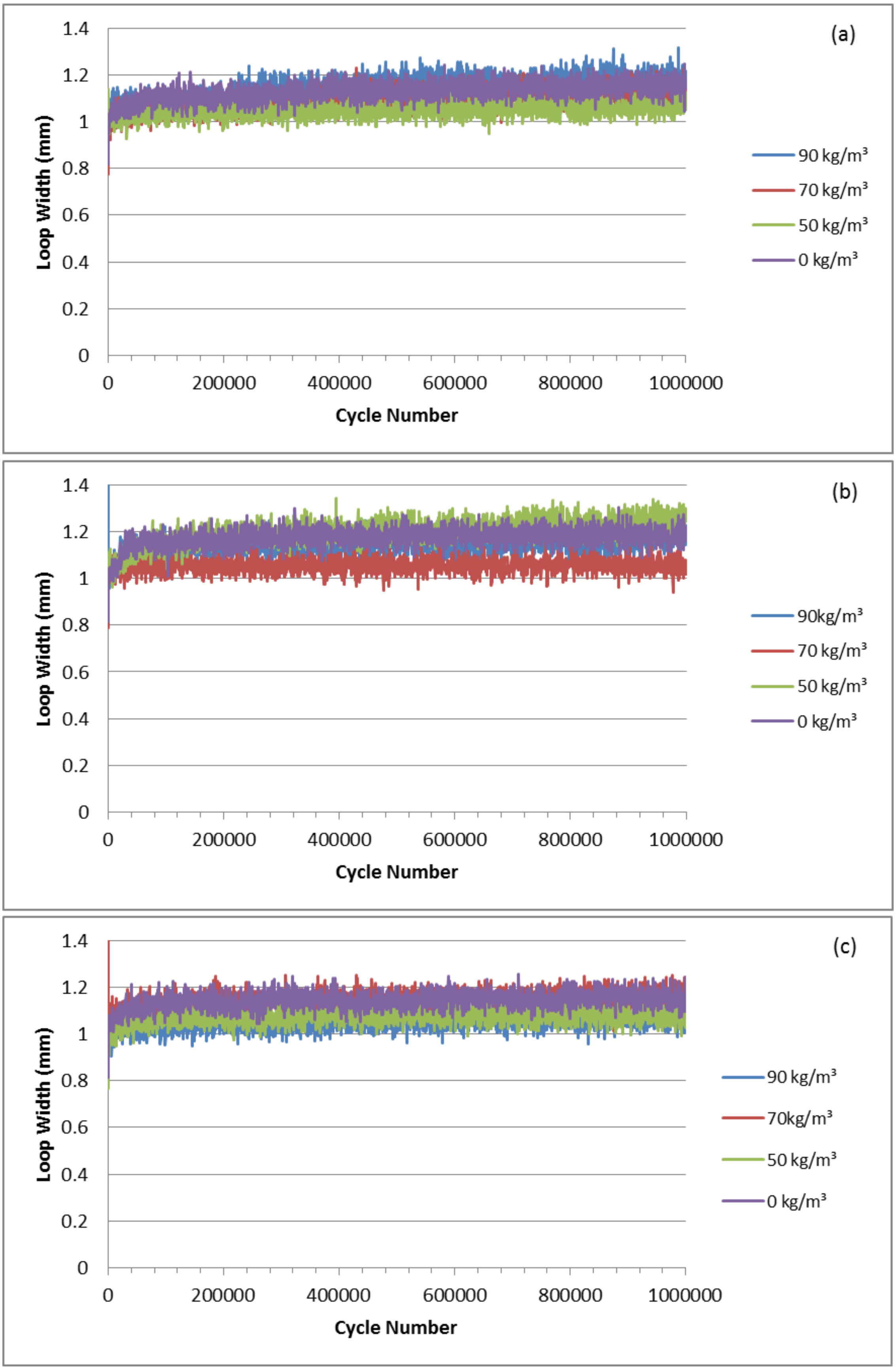

Figure 54: Loop Width for a) 90 MPa b) 70 MPa and c) 50 MPa

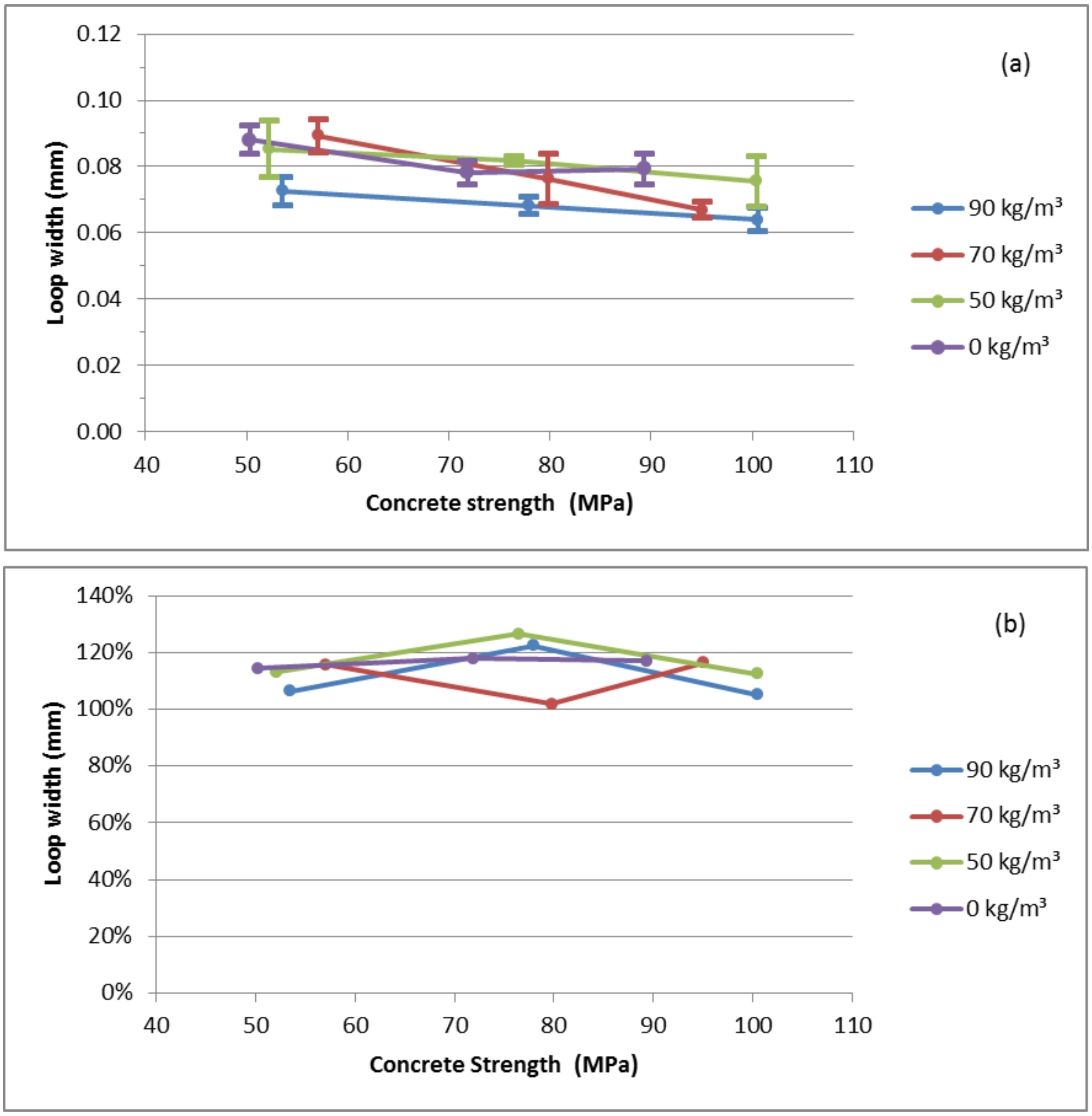

Figure 55: Loop Width a) After 500 Load Cycles, b) Change as % of Initial After 1 000 000 Load Cycles

4.6.8 Cyclic Loading: Hysteresis Energy

Hysteresis energy dissipation can be used to track and predict remaining life (Do et al., 1993). The change in hysteresis energy with increasing load cycles is shown in Figure 56. A moving average of 10 data points has been used as a smoothing function on the data presented in Figure 56 in order to present the trends more clearly. During the cyclic load application little change is observed in the hysteresis energy. It is anticipated that hysteresis energy would initially decrease to a stable point and then rapidly increase before failure is observed. This behaviour is not observed in the results of this experiment. Precracking the samples may have already taken the samples beyond the point where the decrease in energy would be observed. The minimal change in hysteresis energy with continued cyclic loading may also indicate that at 1 million load cycles the concrete is still performing well and is not yet nearing a point of failure.

Comparatively it is seen that hysteresis energy decreases slightly with increasing concrete strength (Figure 57). Increased fibre content is also associated with lesser hysteresis energy although the effect of fibre content is less prominent than that of concrete strength.

After the application of 1 million load cycles this trend has not changed greatly although the distinction between high and low fibre contents has increased (Figure 58). A decrease in hysteresis energy is noted for 50 MPa concrete (Figure 59). The greatest increase is seen in the 70 MPa concrete with 90 kg/m^3 fibre content. Hysteresis energy for the majority of other concrete mixes has remained similar to that at the start of cyclic load application.

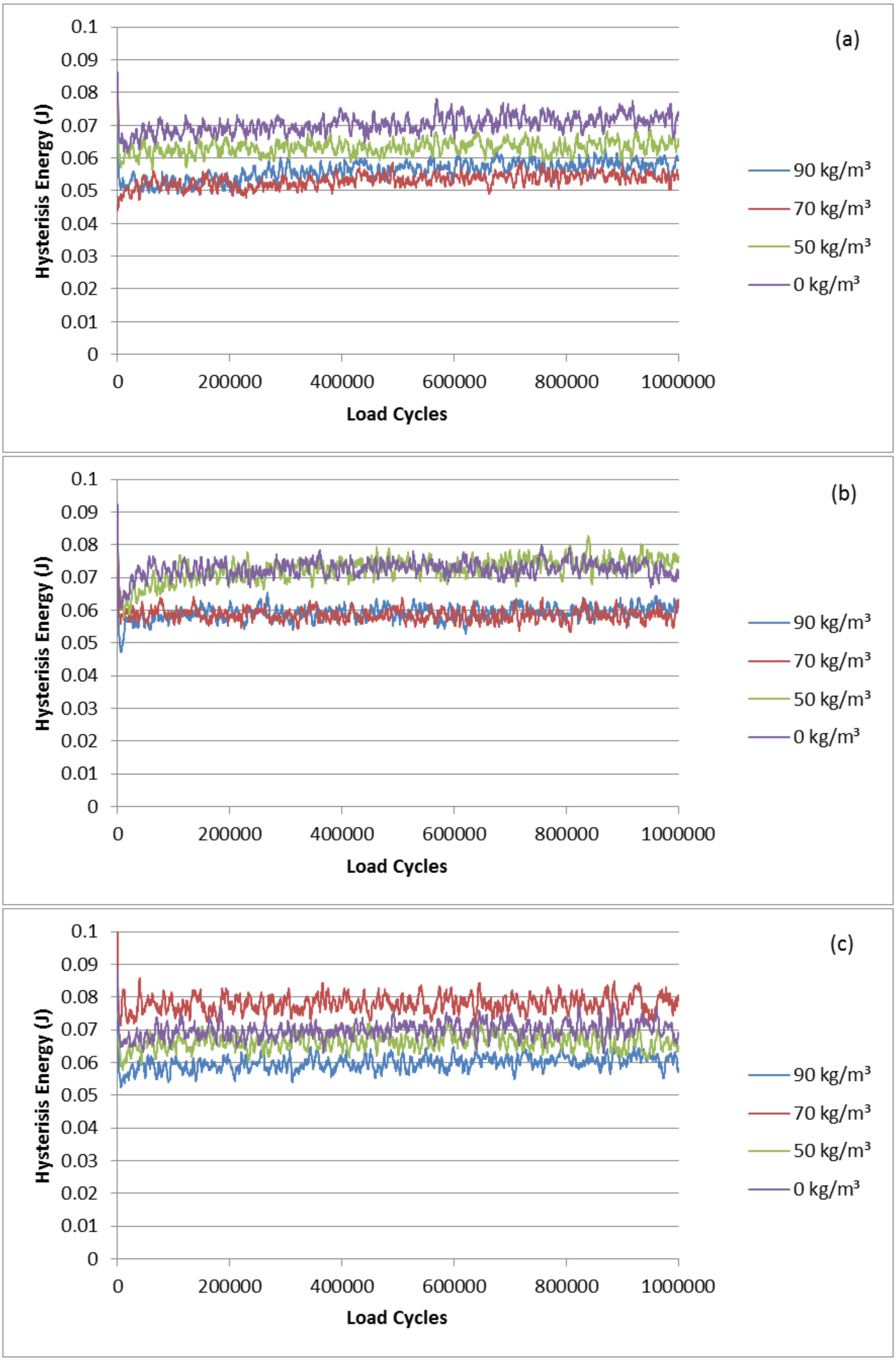

Figure 56: Hysteresis Energy Change with Cyclic Load Application for: a) 90 MPa b) 70 MPa c) 50 MPa

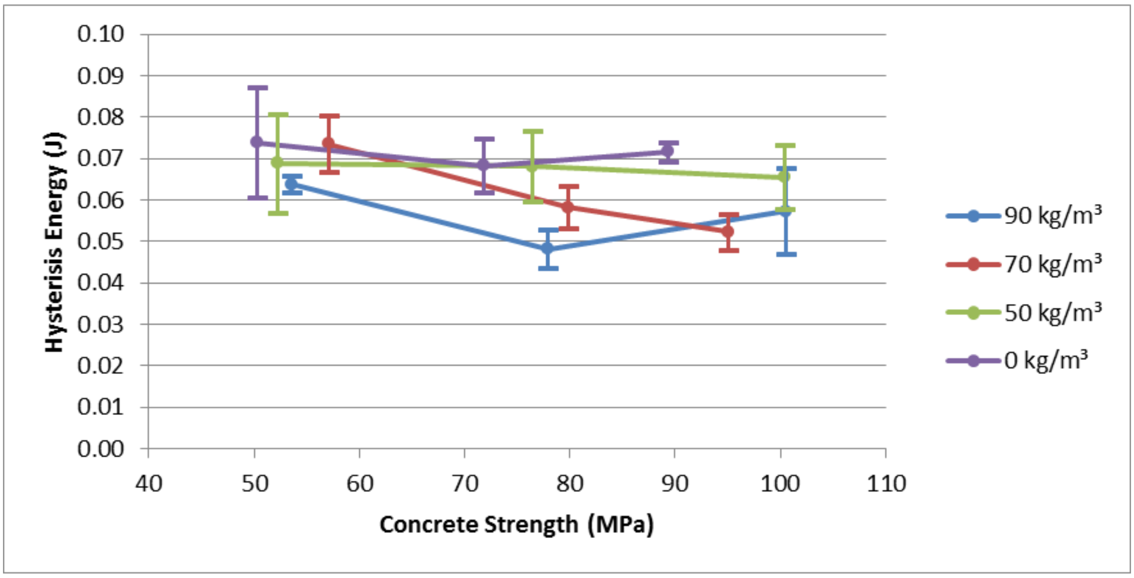

Figure 57: Hysteresis Energy of Slabs at 500 Load Cycles (Error Bars show ±1 Standard Deviation)

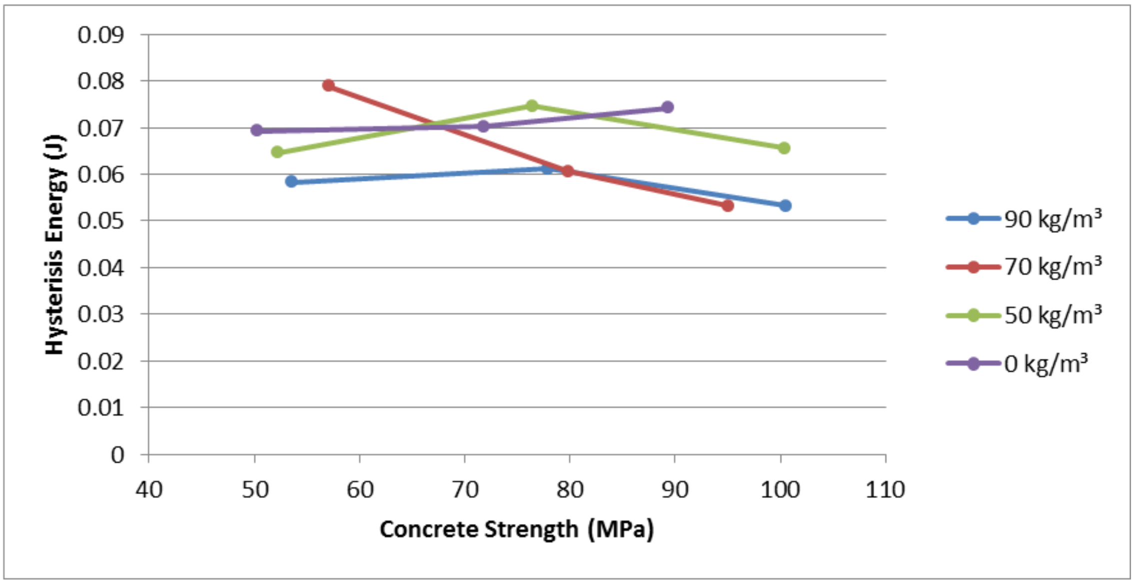

Figure 58: Hysteresis Energy of Slabs at 1 000 000 Load Cycles

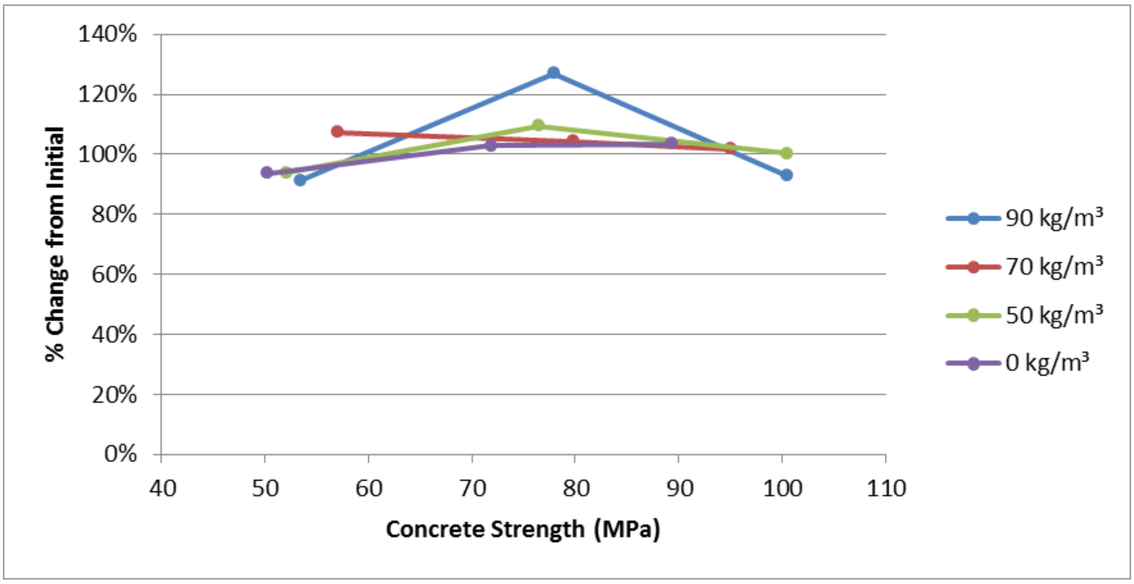

Figure 59: Hysteresis Energy Change in Percentage from 500 Load cycles to 1 000 000 load cycles

4.6.9 Cyclic Loading: Deflection

The third set of data extracted from the measurements taken during the application of cyclic load is deflection. Maximum and minimum deflections are shown in Figure 60. An increase in deflection is seen with increasing load repetitions. Initially there is rapid increase in deflection; however, this rate soon decreases. As expected, high strength concrete with high fibre content initially has the lowest maximum deflections (Figure 61a). After the application of one million cycles samples were seen to have similar maximum deflections. Samples without fibres however had the highest deflections. Among samples containing fibres maximum deflection is noted to decrease with increasing concrete strength (Figure 61b). When compared to the initial maximum deflection the greatest increase is seen on the samples of 90 MPa concrete with 90 kg/m³ of fibres (Figure 61c). Low concrete strength samples with low fibre contents tend to show greater increases.

It should be reiterated that the deflection values measured here are slab deflections over a span of 450 mm, therefore these deflections do not represent deflections that can be expected on a road pavement. The comparative measurements do however give an indication of the fatigue resistance of the material and thus which concrete mix is likely give the best pavement performance.

While the general trend is a gradual increase in deflection, there are certain intervals where the deflection increases rapidly. These “steps” or regions of rapid increased deflection appear to occur at regular intervals. These steps also tend to occur after approximately the same number of load repetitions. This stepping behaviour was not seen in the stiffness data, loop width data or hysteresis energy data. (Figure 52, Figure 54 and Figure 56). These steps occur at an interval of 250 000 cycles. At a loading rate of 3 Hz this would equate to a time interval of approximately 24 hours.

Without fibres all the samples perform poorly, but the inclusion of even 50 kg/m³ fibres greatly improves the deflection performance (Figure 61b). Increasing the fibre content from 50 kg/m³ to 90 kg/m³ does not appear to enhance maximum deflection performance. While the steel fibres are clearly important, deflection appears not to be particularly sensitive to fibre content. The dominating factor controlling deflection appears to be concrete strength.

Recoverable deflection is the difference between maximum and minimum deflection measured during application of cyclic loading. There is an initial rapid increase in recoverable deflection which then stabilises (Figure 62). The steps seen in the deflection measurement are not observed here. The implication of this is that the minimum and maximum deflections increase in a synchronised manner.

Initial recoverable deflection is lower for samples with higher fibre contents but appear not to be significantly influenced but concrete strength (Figure 63a). The change in recoverable deflection after one million load cycles is shown as a percentage of the recoverable deflection at 500 load cycles in Figure 63b. The increase is in the order of 110% with a maximum of 120%.

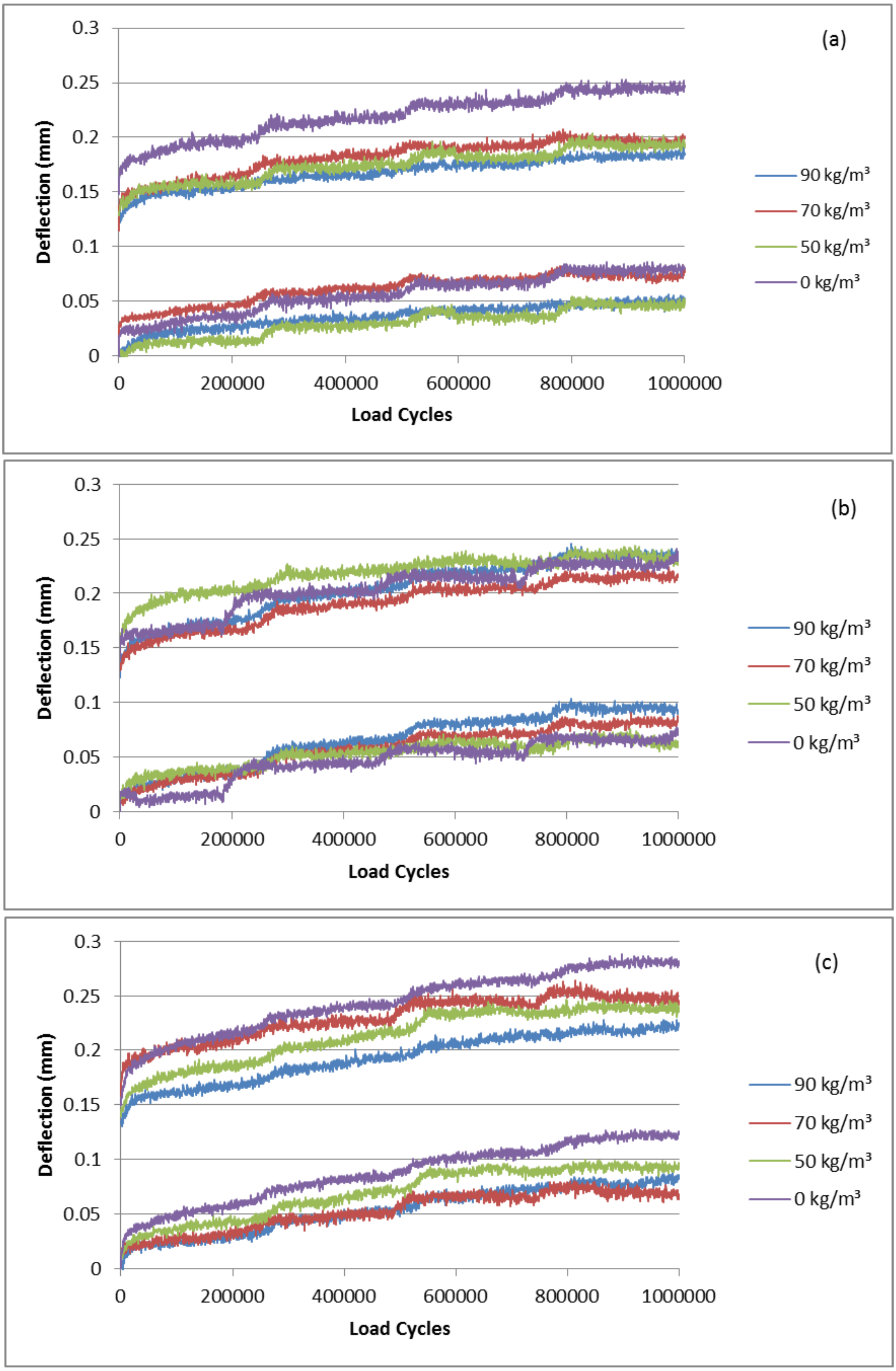

Figure 60: Cyclic Loading Deflection for a) 90 MPa b) 70 MPa and c) 50 MPa

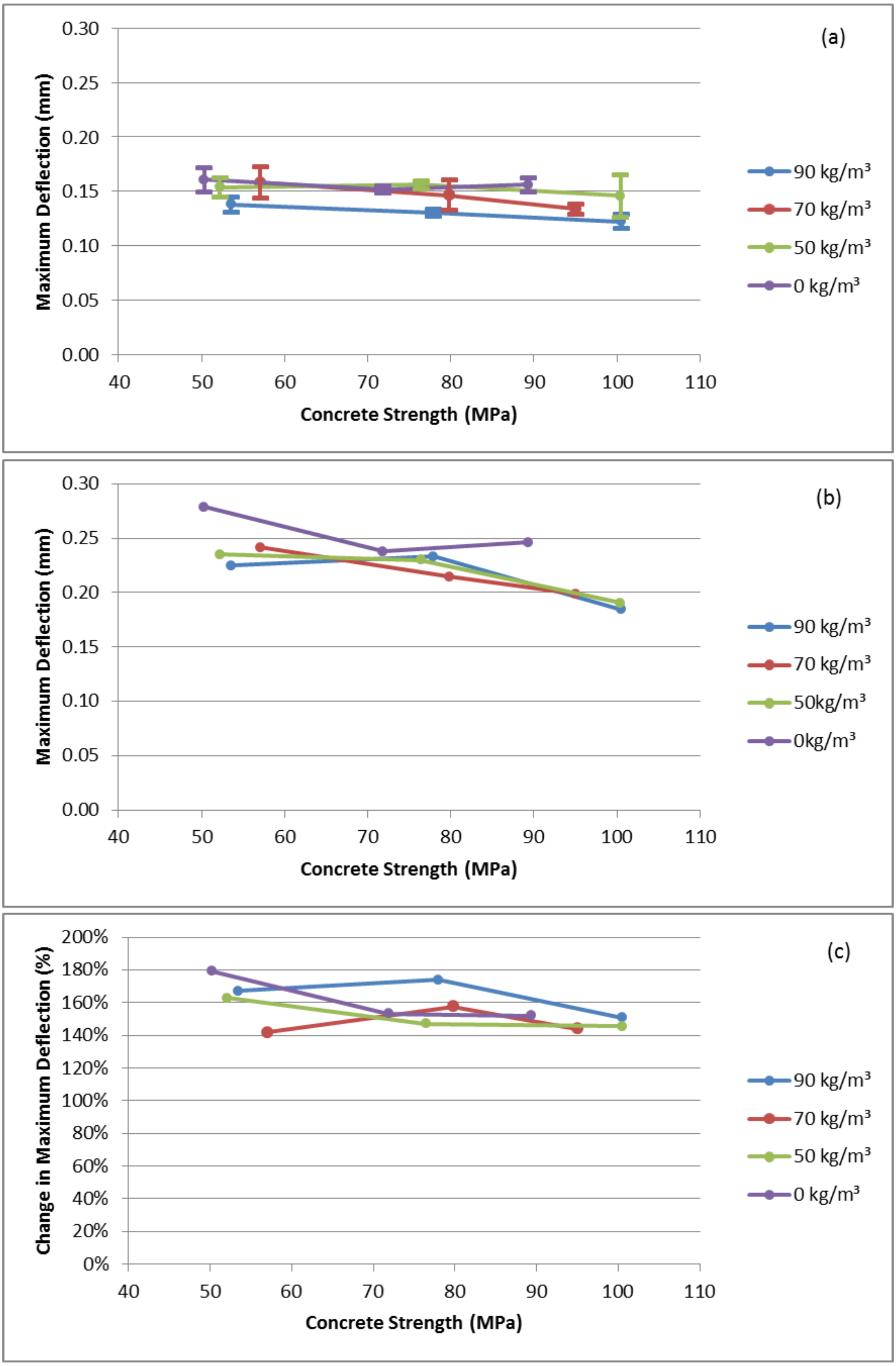

Figure 61: Maximum Deflection: a) at 500 Load Cycles, b) at 1 000 000 Load Cycles, c) % Increase from 500 to 1 000 000 Load Cycles

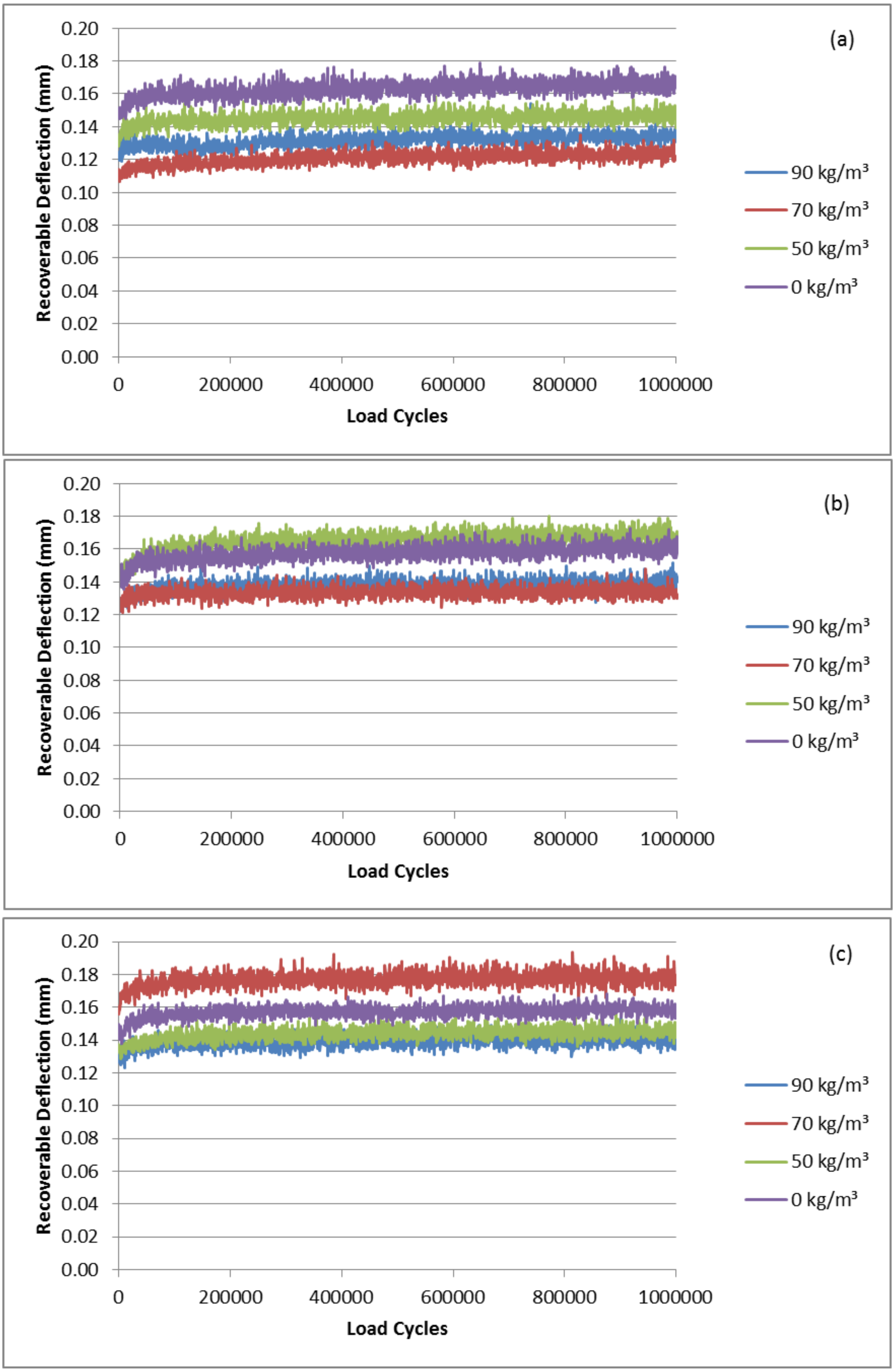

Figure 62: Recoverable Deflection: a) 90 MPa, b) 70 MPa, c) 50 MPa

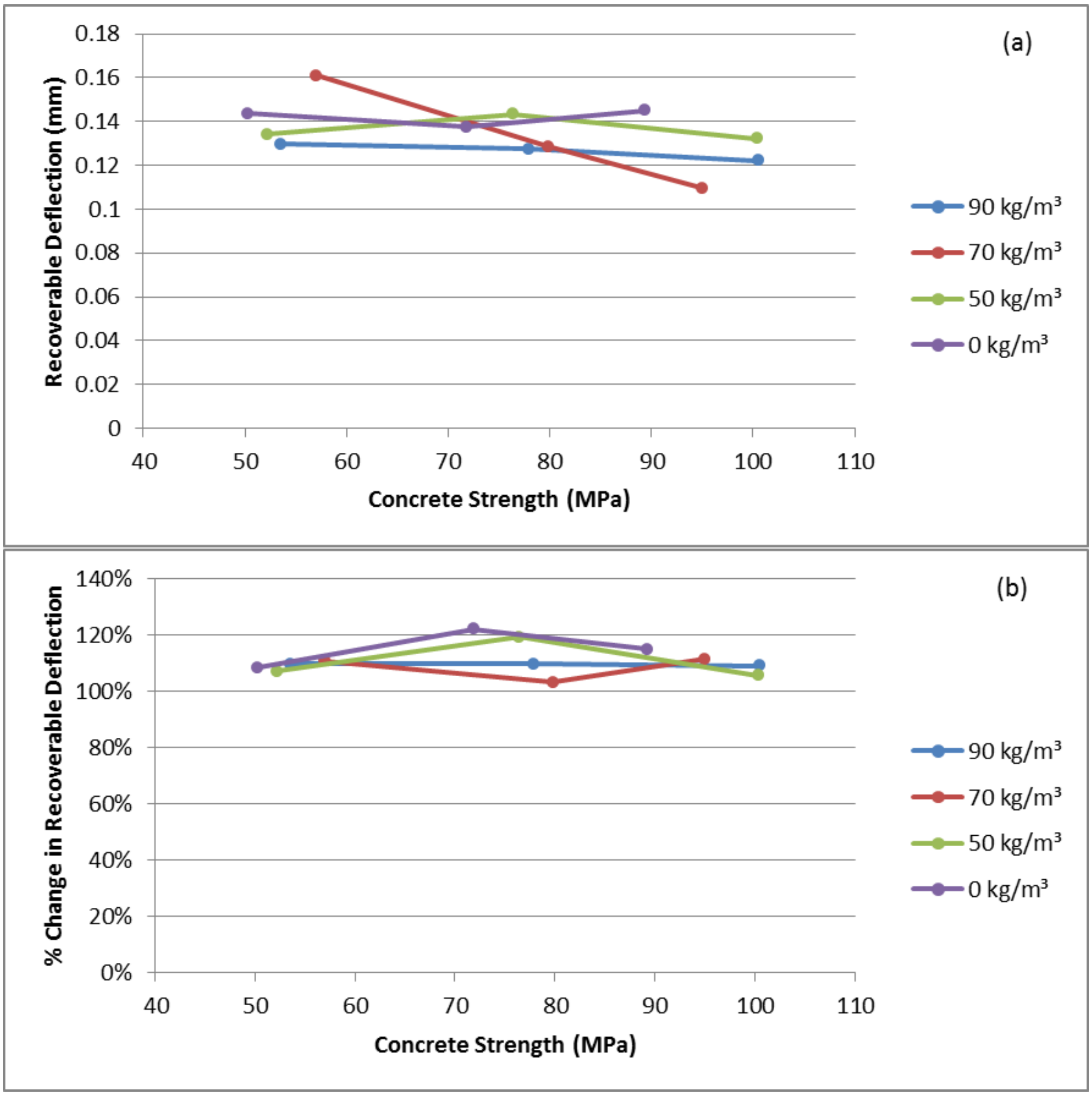

Figure 63: Recoverable Deflection: a) at 500 Load Cycles, b) % Change from Initial Value to Value at 1 000 000 Load Cycles

4.6.10 Cyclic Loading: Conclusion

It can be concluded that data gathered during the application of cyclic loads provides useful performance measures that make for good comparative measures between samples. A better indication of fatigue performance was obtained by this testing method than from the monotonic load results. Stiffness, loop width, hysteresis energy and deflection results all proved to be good and useful measures.

5 Conclusions and Recommendations

5.1 Introduction

From the research conducted to date it is shown that UTCRCP has very good prospects as a road pavement option. It is able to carry high traffic loads for long periods of time while requiring little maintenance over its life. The combination of these two factors is required to give UTCRCP a competitive life cycle cost.

However, the complex material interactions are not yet fully understood. Certain phenomena observed on the trial sections, constructed on South African roads, have revealed some undesirable behaviour that may make designers and road authorities reluctant to implement UTCRCP.

5.2 Findings: Effect of Concrete Strength and Fibre Content

The first objective of this research was to measure the effect of changing fibre content and concrete strength on the material properties of fibre reinforced concrete. The effects of changing these parameters have been demonstrated and discussed in Chapter 4. It was clearly demonstrated that changing concrete strength and fibre content has a large effect on the mechanical performance of UTCRCP. The interactions and effects of these two variables are complex and depending on the intended application of the concrete different mixes will be optimal. As this decision will influence both the lifespan and cost, concrete strength and fibre content must be selected with care.

It is generally assumed that better performance will be achieved by increasing concrete strength and increasing fibre content. The results of the experiments of this study show that this is not true. For example if the tensile performance is considered as was measured by MOR, fibres have almost no effect. However, if the post-crack performance as measured by strain energy is considered, fibre content has a large effect. Should the flexural energy absorption be identified as the critical performance measure the optimum mix would be high strength concrete with a high fibre content in a low deflection environment. In a high deflection scenario greater energy absorption can be obtained with 50 MPa concrete with no fibres.

A further example is that of hysteresis energy. After one million cycles concrete with fibre contents of 70 kg/m^3 and 90 kg/m^3 have similar performance, while concrete with a fibre content of 50 kg/m^3 is similar to concrete that has no fibres. If minimising hysteresis energy was the deciding performance factor then a fibre content greater than 70 kg/m^3 must be used if any mechanical advantage is to be gained.

For certain mechanical properties different concrete mixes produce better results. There is not one mix that is best in all scenarios. Depending on which mechanical property, or properties, the designer deems most important a selection of fibre content and concrete strength can be made.

5.3 Findings: Regarding Material Behaviour and Test Methods

The second objective of this research was to use the results obtained to compare the test methods in order to comment on their effectiveness and sensitivity to concrete strength, fibre content and test method.

After the series of laboratory tests, the findings on these aspects are presented here.

Basic material properties:

- Cube strength is a poor measure of fibre content. Thus cube strength can be used as a quality control test for the concrete but should not be expected to give any information regarding the quantity or effectiveness of the fibres.
- The standard splitting cylinder test is a poor measure of the fibre performance. The splitting cylinder test can be used as a quality control test for the concrete but should not be expected to give any information regarding the quantity of effectiveness of the fibres. However, if the lateral displacement is measured during the test the effect of the fibres can be measured.
- Fibre reinforced concrete has non-linear elastic compressive behaviour.
- Elasticity can be related to initial stiffness and ultimate compressive stress.
- Poisson’s ratio is not constant and is affected by concrete strength but is not sensitive to fibre content, except at lower concrete strengths.

Slabs under monotonic loads:

- The MOR in slabs is a good measure of the concrete strength but is not sensitive to fibre content. Splitting cylinder test results correlate well with MOR of the slabs, although splitting cylinder strength under-predicts the MOR of the slabs.
- The MOR of slabs can be used as a design or quality control measure, but these results show that MOR is a measure of the concrete and is not an appropriate measure of the fibre content or performance.
- Maximum load is a poor indicator of performance. From these results it would appear that it is insensitive to concrete strength and fibre content. It can be argued that maximum load is not a relevant parameter for pavement design. It is recommended that maximum load is not used as a quality control measure for UTCRCP.

- Samples tested monotonically after the application of cyclic loading display little drop in maximum load. It is thus concluded that this test is not a useful measure for fatigue damage. However at higher fatigue stresses or higher number of fatigue load cycles it may give better results.

Energy absorption: monotonic loading

- The energy absorption must be used with caution. It was found that lower strength concrete can absorb more energy at large deflections. The deflection limit, up to where energy absorption is calculated, needs to be specified carefully. This limit needs to be set at a value that will fairly evaluate the performance of the concrete in its intended function.
- A more appropriate deflection should be selected for calculation of energy absorption when testing concrete for road pavement applications.

Slabs under cyclic loading loads:

- Testing maximum load monotonically after cyclic load application is not an effective measure of fatigue performance.
- Testing slab stiffness monotonically after application of cyclic load does give a good measurement of material damage. However this method requires more samples to be prepared.
- Measuring load and deflection during cyclic load application does give good measures of performance. The advantage of this method is that the decrease in performance can be tracked over regular load repetition intervals. Additionally fewer samples are required, as the damage is tracked continuously through the test.
- Properties that can be determined from load deflection data recorded during the application of cyclic loading include: stiffness, loop width, hysteresis energy, maximum deflection, minimum deflection and recoverable deflection. These methods all give good measures of material damage and have the advantage of recording the change in behaviour as the cyclic loading continues.

Measuring load deflection data during the application of cyclic load application is an effective method of measuring the fatigue damage to concrete. This data allows for the calculation of stiffness, loop width, hysteresis energy, maximum deflection, minimum deflection and recoverable deflection. With the results available from these experiments it would appear that the measurements of these tests are repeatable. This method of testing can be recommended to be used in future for testing fatigue performance of fibre reinforced concrete.

These tests are useful measures of UTCRCP fatigue performance, however they are time consuming. For this reason they are better suited to research and initial design tests for UTCRCP. If these results could in future be linked to both pavement performance and static test results it would be possible to use predicted fatigue performance as part of a quality control of UTCRCP.

5.4 Findings: An Optimised Concrete Mix Design for UTCRCP

The third objective of this experiment was to use the results of the tests in order to recommend an optimum concrete design. A summary of results of the slab tests are given in Table 8. All the results of the slab testing are summarised regardless of how relevant or effective the test was found to be. For ease of comparison the figure where the results are presented is given in the last column of the table.

Looking at the overall trend in results it would appear that 90 MPa concrete gives the best performance. The optimum fibre content would seem to be 70 kg/m³ although a fibre content of 90 kg/m³ in some cases does give more favourable performance. Depending on which test method the designer selects, the most critical performance factor for UTCRCP an optimum mix design can be selected.

A possible reason that the 70 kg/m³ may outperform the 90 kg/m³ may be attributed to compaction. The addition of fibres reduces the workability of the concrete; this in turn increases the difficulty of compacting the concrete. However there is not sufficient data from this experiment to give a definitive answer regarding optimum concrete strength and fibre content.

Table 8: Summary Of Slab Test Results

Load condition of Samples	Test	Best performing Concrete Mix	Figure
Samples tested under monotonic load in Uncracked state	Maximum load	70 MPa 70 kg/m³	Figure 40
	Energy absorption 1.8 mm	90 MPa 90 kg/m³	Figure 43a
	Energy absorption 10 mm	70 MPa 70 kg/m³	Figure 43b
	Energy absorption 15 mm	50 MPa 50 kg/m³ *	Figure 43c
Samples tested under monotonic load after Precracking and application of cyclic load	Energy absorption 1.8 mm 1 000 000 Load cycles	90 MPa 90 kg/m³	Figure 49a Figure 46
	Energy absorption 10 mm 1 000 000 Load cycles	50 MPa 90 kg/m³	Figure 49b
	Energy absorption 15mm 1 000 000 Load cycles	50 MPa 90 kg/m³ *	Figure 49c
	Stiffness/ Slope to MOR (static)	90 MPa 70 kg/m³	Figure 50
Samples tested during application of cyclic load after Precracking	Stiffness with cyclic loading	90 MPa 70 kg/m³	Figure 53 b)
	Loop width	70 MPa 90 kg/m³ or 90 MPa 90 kg/m³	Figure 55 b)
	Hysteresis energy	70 MPa 90 kg/m³ or 90 MPa 90 kg/m³	Figure 58
	Maximum Deflection (cyclic loading)	90 MPa 90 kg/m³	Figure 61b
	Recoverable Deflection	90 MPa 70 kg/m³	Figure 63

*measurements at large deflections

5.5 Recommendations

Optimised mix design may contribute to a more economic UTCRCP design. Better insight into the following matters would be beneficial to the successful future implementation of UTCRCP:

- Further experimentation using the fatigue methods used here to test performance at higher numbers of load repetitions.
- Effect of span length for slab testing fatigue (linked to a realistic radius of curvature).

5.6 Closing Remarks

This research shows that by increasing fibre content and concrete strength a better fatigue performance is not necessarily achieved. A more economical design may be possible. Furthermore a mix design with a lower concrete strength and fibre content may offer better performance.

Comparative results of a suite of laboratory control tests are shown indicating the advantages and limitations of these tests. The effect of changing fibre content or concrete strength on the mechanical properties of the concrete is also shown. UTCRCP is a promising paving technology that offers some attractive benefits. Further research is required to improve and economise the design, as well as better predict the expected life span of the pavement.

The importance of selecting the correct test method and interpreting the results is highlighted. This is especially important when deflection energy absorption is used. Selecting a deflection limit to which energy absorption should be evaluated is crucial.

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Appendix A

Presented here is data supplementary to the data presented in Section 4.6.3-Energy Absorption-Without Cyclic Loading

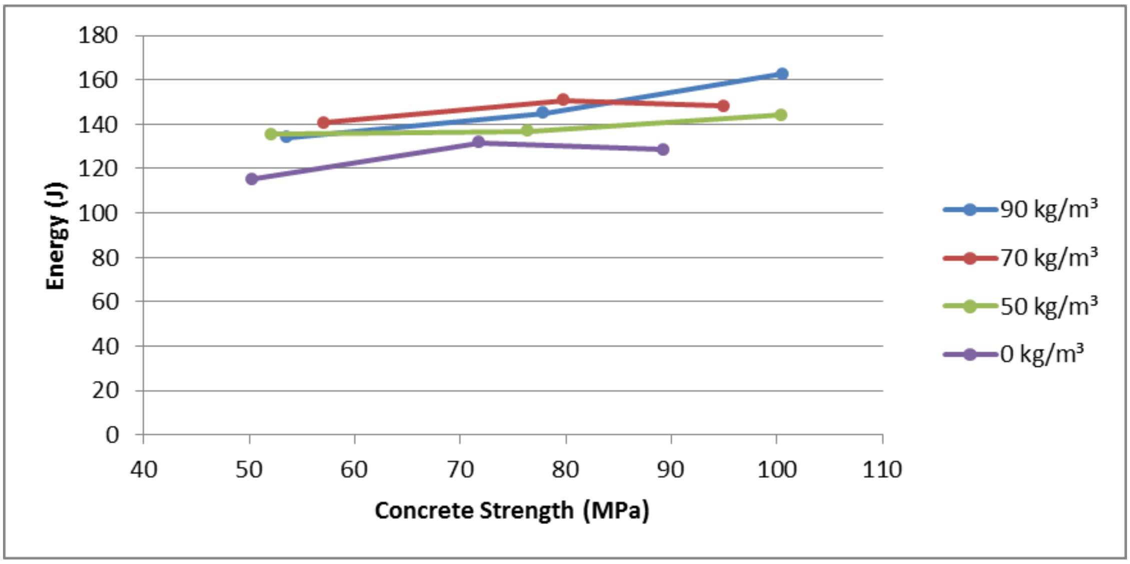

Figure A-1: Energy Absorption at 5mm Deflection - Un-cracked Sample

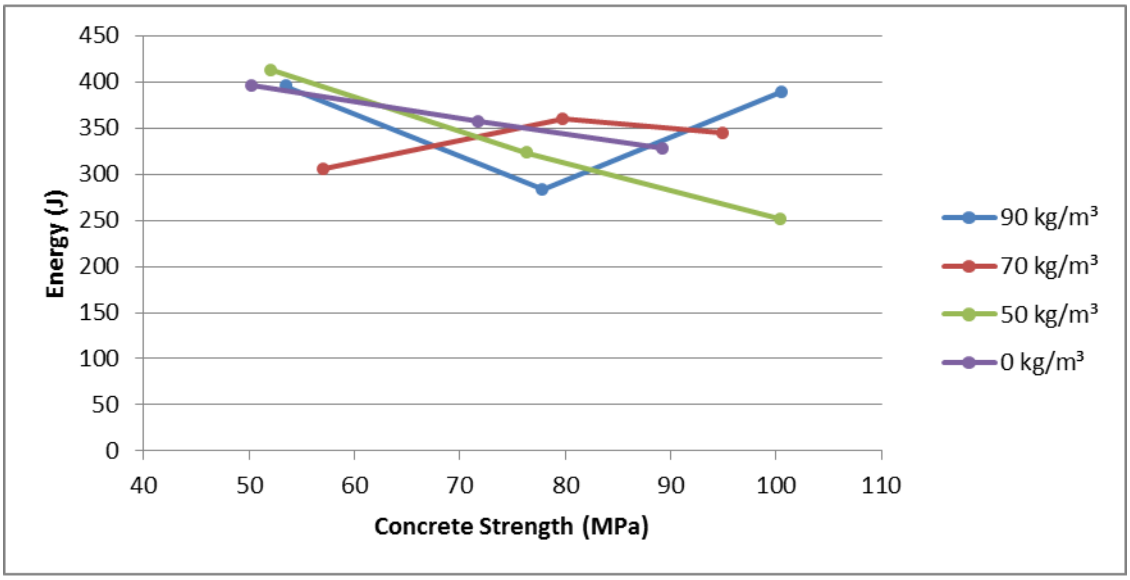

Figure A-2: Energy Absorption at 25mm Deflection - Un-Cracked Sample

Appendix B

Presented here is data supplementary to the data presented in Section 4.6.4 -Energy Absorption- With Cyclic Loading

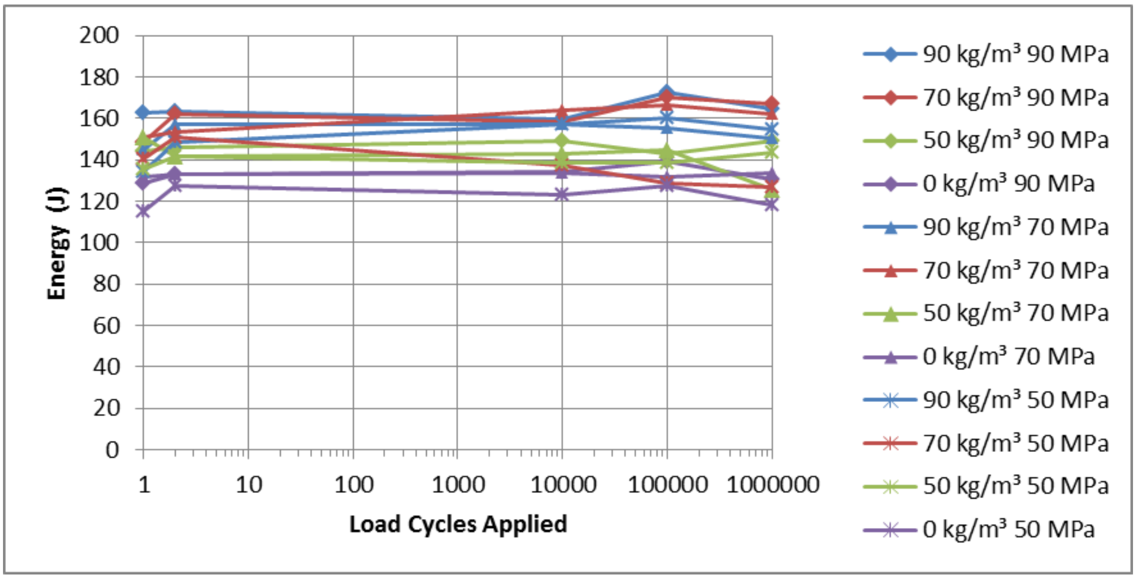

Figure B-1: Energy Absorption to 5mm Deflection with Cyclic Loading

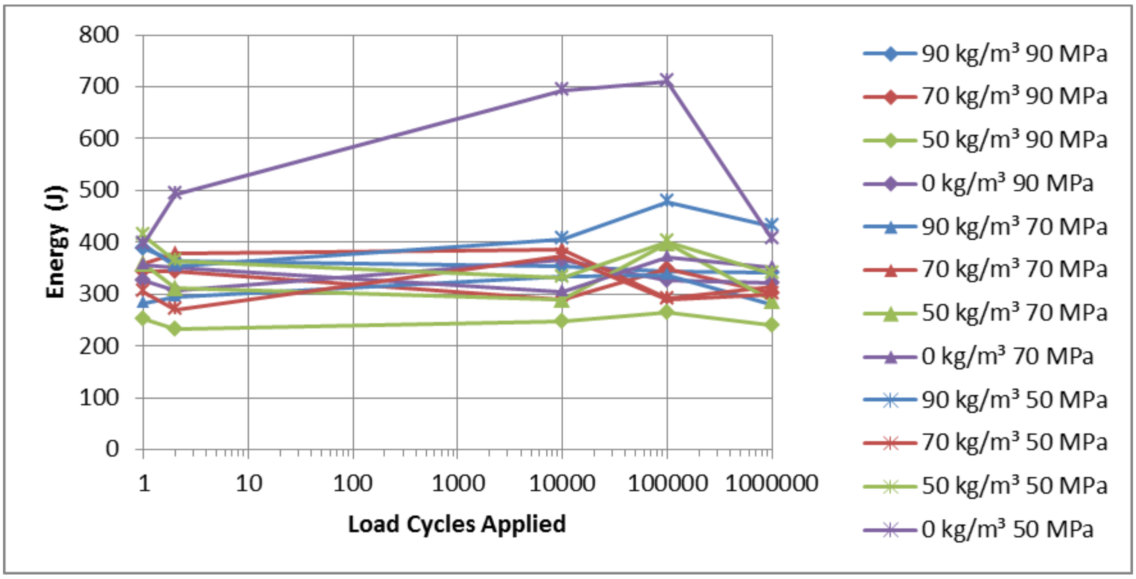

Figure B-2: Energy Absorption to 25mm Deflection with Cyclic Loading

Appendix C

Presented here is data supplementary to the data presented in Section 4.6.4 -Energy Absorption- With Cyclic Loading

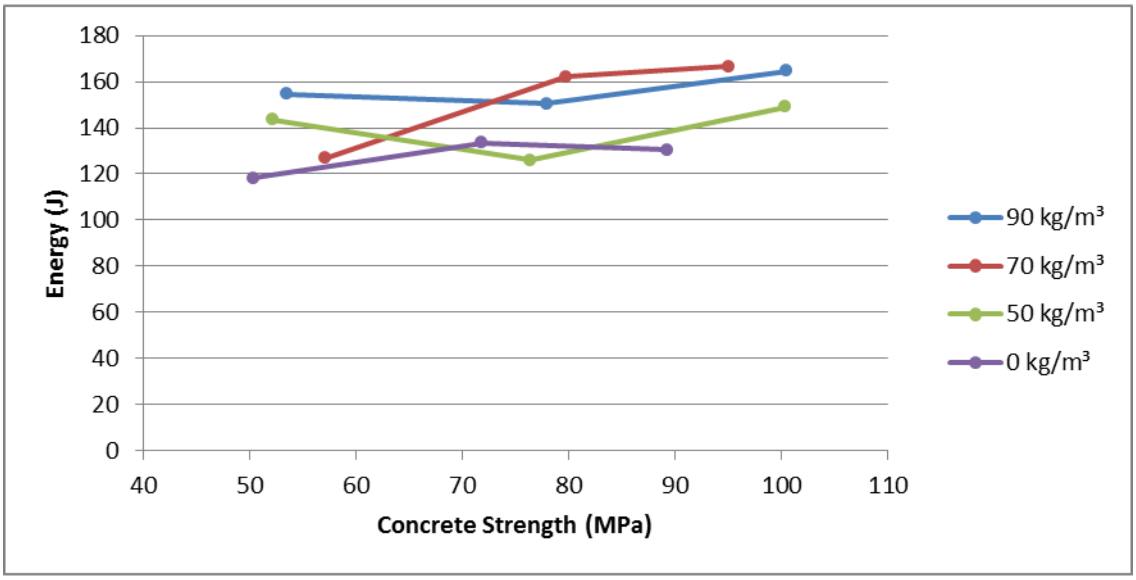

Figure C-1: Energy Absorption at 5mm Deflection of Samples Tested Monotonically After 1 000 000 Load Applications

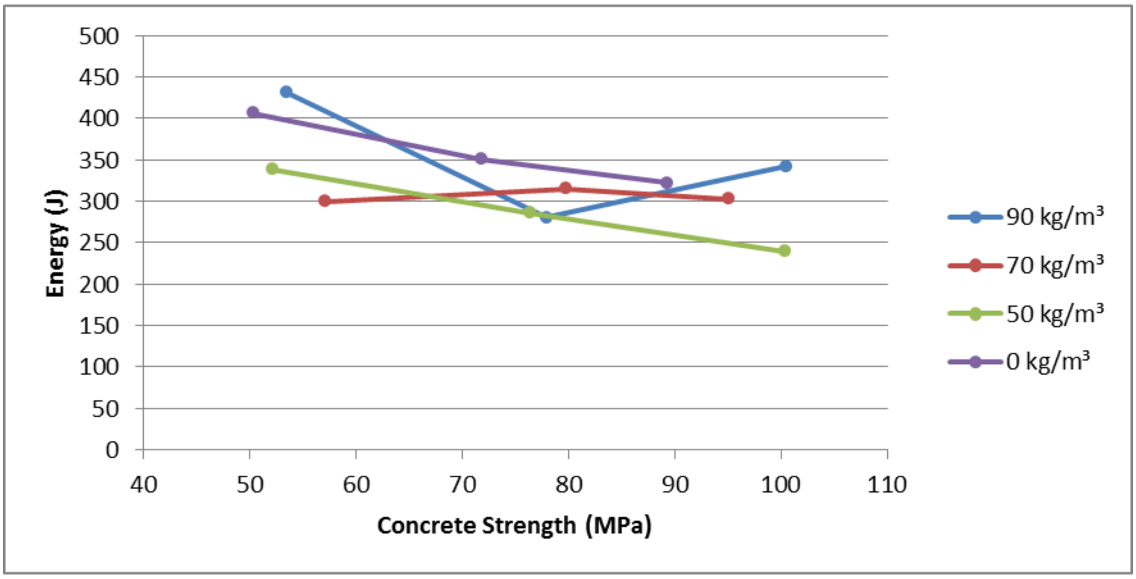

Figure C-2: Energy Absorption at 25mm Deflection of Samples Tested Monotonically After 1 000 000 Load Applications

Appendix D

Presented here is data supplementary to the data presented in Section 4.6.6 -Cyclic Loading: Stiffness

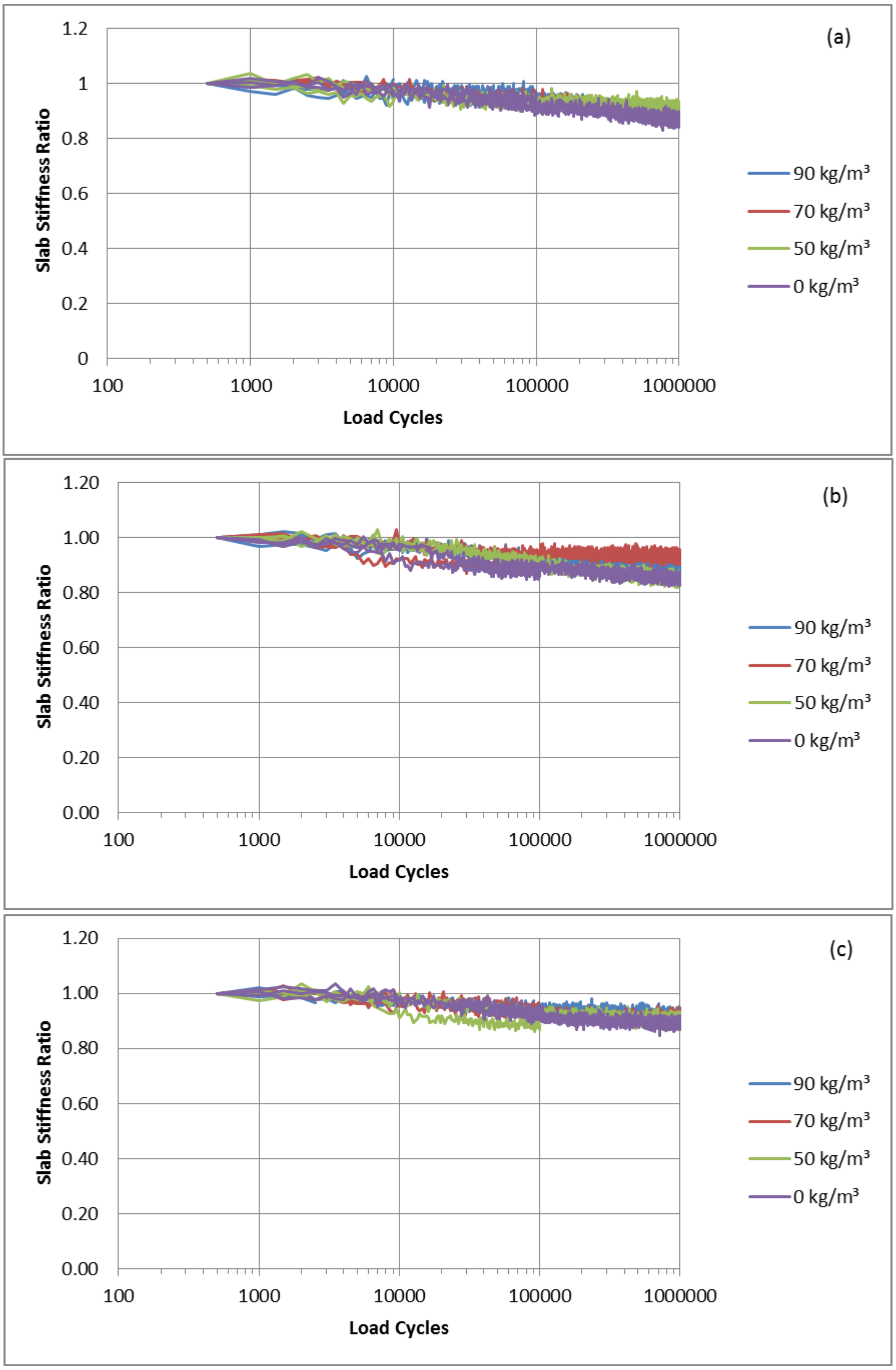

Figure D-1: Slab Stiffness as Ratio of Initial Slab Stiffness -Log Scale- for: a) 90 MPa b) 70 MPa and c) 50 MPa