CHARACTERISATION AND REFINEMENT OF PROPERTIES OF GLASS FIBRE REINFORCED POLYESTER POLYMER CONCRETE FOR USE IN MANHOLE COMPONENTS

By

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ABSTRACT

The aim of this investigation is to characterise and refine the physical properties of glass fibre reinforced polyester polymer concrete. This material is currently being employed by AV Mouldings (Pty) Ltd. to manufacture manhole and drain components according to specifications existing for *cast iron* covers. No specification exists for polymer concrete. In particular it has been found that there is a large market for Type 2A replacement manhole covers and frames due to the current problem in South Africa of the cast iron versions being stolen and sold for scrap metal. It has been found that polymer concrete covers manufactured to replace stolen cast iron covers (in existing cast iron frames) fail occasionally in service.

The investigation thus focuses on the characterisation of glass fibre reinforced polymer concrete and analysis of the current standards with a view to establishing a new South African Bureau of Standards (SABS) specification for polymer concrete manhole components. The main testing procedure involved flexural testing of beam specimens. Preliminary tests were carried out to measure strength, toughness, strain rate sensitivity, and the effect of different reinforcing materials. Accelerated degradation tests were then conducted to establish the materials resistance to UV radiation, acids, alkalis, and various solvents. Different resins were evaluated, and experiments were conducted using graded aggregates, in an attempt to reduce the number of voids in the material. Vibratory moulding techniques and postcuring methods were also evaluated. The viability of employing silane coupling agents in polymer concrete was investigated in detail towards the end of the research. Redesign of the Type 2A replacement cover was then undertaken.

It is found that polyester polymer concrete is susceptible to degradation by acids, alkalis, saline solutions and water. Alkaline solutions are found to be the most aggressive. It has been established that a certain amount of UV degradation is possible, but is likely to be confined to surface regions. Graded aggregates seem to be detrimental to the mechanical properties of polymer concrete, due to the fact that mainly angular particles were used. Postcuring procedures also do not improve the properties, but it has been found that silane coupling agents work well in this type of polymer concrete. Vibration methods during moulding have a detrimental effect on the flexural strength of specimens due to segregation of the aggregates and the resin. Void removal is hindered by the side walls of the mould – it is thus thought that in larger mouldings, vibration methods may be beneficial.

Stress analysis of the Type 2A replacement cover reveals that it is the change in section thickness and the small radius in the lip region that is the cause of failure. This stress concentration coupled with the relatively low fracture toughness of polymer concrete results in circumferential lip cracks that propagate through the material with each cycle of loading in service. Redesign of the cover has been undertaken, and a few possible solutions are assessed.

The evidence suggests that manhole covers and frames can be successfully manufactured from polymer concrete, and for this reason, recommendations will be made to the SABS for specifications regarding these components.

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List of symbols and abbreviations

PC	polymer concrete
SABS	South African Bureau of Standards
DCPD	dicyclopentadiene
ortho	orthothphalic polyester resin
iso	isothphalic polyester resin
ϕ	diameter
a	radius, long dimension, notch depth
b	width or short dimension
w, t, d	thickness dimension
r	radius variable
z, y	thickness variable
W	distributed load
E	Young's modulus
ν	Poisson's ratio
σ	stress (bending)
δ	plate deflection
M	applied bending moment
I	second moment of area
MOR	modulus of rupture
K_{1C}	fracture toughness
Y	geometrical factor
θ	angle of crack propagation
Ave, $\bar{x}$	average of sample
STD, s	standard deviation
μ	mean of population
n	size of sample (no. of specimens)
F	applied force
L	span of three point flexure test
T	time (period)
V	speed

TABLE OF CONTENTS

1. INTRODUCTION

Polymer concrete, or resin concrete has been used as an engineering material for decades. However, it is only recently that it has been used for load carrying applications. The reason for this is that its behaviour has not been well understood, and limited research work has been conducted regarding this interesting material. Essentially, polymer concrete consists of a polymer resin system combined with an aggregate mixture. Certain other additions are common, eg. pigments, chopped strand fibres, woven roving reinforcement, and coupling agents etc. In particular, the addition of fibre reinforcement into traditional polymer concrete enhances the materials load carrying capacity considerably, and thus the material has become more common for engineering purposes.

The work undertaken in this research project was requested by AV Mouldings (Pty) Ltd., a subsidiary company of the Power Group in the Western Cape, South Africa. AV Mouldings currently manufacture manhole and drain components from polymer concrete. The market for these products has increased considerably over the last few years due to the problem of cast iron covers being stolen and sold for scrap.

The AV Mouldings product range includes various components described by SABS 558 : 1973 “Cast iron surface boxes and manhole and inspection covers and frames”, and SABS 1115 : 1976 “Standard specification for cast iron gratings for gullies and storm water drains”. In particular, the Type 2A heavy duty cover detailed in SABS 558 : 1973 (see Appendix C) commonly seen on South African roads is in high demand.

The problem with these products is that though they are manufactured according to SABS specifications, they cannot bear the SABS mark, since these specifications exist solely for cast iron covers and frames. Furthermore, no SABS specification even exists for polymer concrete manhole and drain components. Another problem is that although AV Mouldings have been in business for at least three years, their knowledge of how the material behaves in service is limited.

The Type 2A "replacement cover", manufactured to replace stolen cast iron 2A covers (in existing cast iron frames) also has design flaws that have been resulting in the failure of some components in service. AV Mouldings have designed and manufactured a *new* Type 2A cover and frame which has been performing well in service. These covers and frames can only be used in cases where a new frame can be installed, since the Type 2A replacement cover design is limited by the shape of the cast iron frame that it must fit into.

Therefore, the objectives of the research undertaken are: -

1) To catalogue and analyse any previous failures or problems.
2) To conduct accelerated degradation tests on materials.
3) To investigate the possibility of varying the aggregate size in the mix.
4) To assist in redesign of replacement covers to fit existing frames.
5) To examine the effect of using different resins, from different suppliers.
6) To investigate the possibility of using postcuring procedures.
7) To investigate vibratory moulding methods.
8) To assist in the incorporation of products into an SABS specification.

There were certain other objectives relating to the SABS approval of products, but due to the time required for the SABS to draw up the specification, it was accepted that this part of the project would be unachievable in the available time. However, regular contact has been made with the SABS head office in Pretoria, and they have agreed that a new specification for manhole covers and frames is to be drawn up. Recommendations regarding a new specification are included in this report in Appendix B.

The methods employed to achieve the objectives set out included extensive mechanical testing of specimens (mainly in flexure mode), accelerated degradation tests involving exposure to aggressive environments, and also exposure to UV radiation. It was clear from the start that individual results are subject to a fair degree of statistical variation, and thus statistical methods are used to evaluate results.

It is also stressed here that the report includes the results of tests on a polymer concrete system based on the standard material used by AV Mouldings. Exotic systems are not covered by this investigation. Conclusions are drawn based on the advantages and disadvantages of various changes made to the mix, and the financial implications of variations in the material are also taken into account. It should also be mentioned that since the research was originally requested by AV Mouldings, the conclusions and recommendations made in this report apply to their operation specifically. Certain results may be applicable to other operations depending on the particular manufacturing process used.

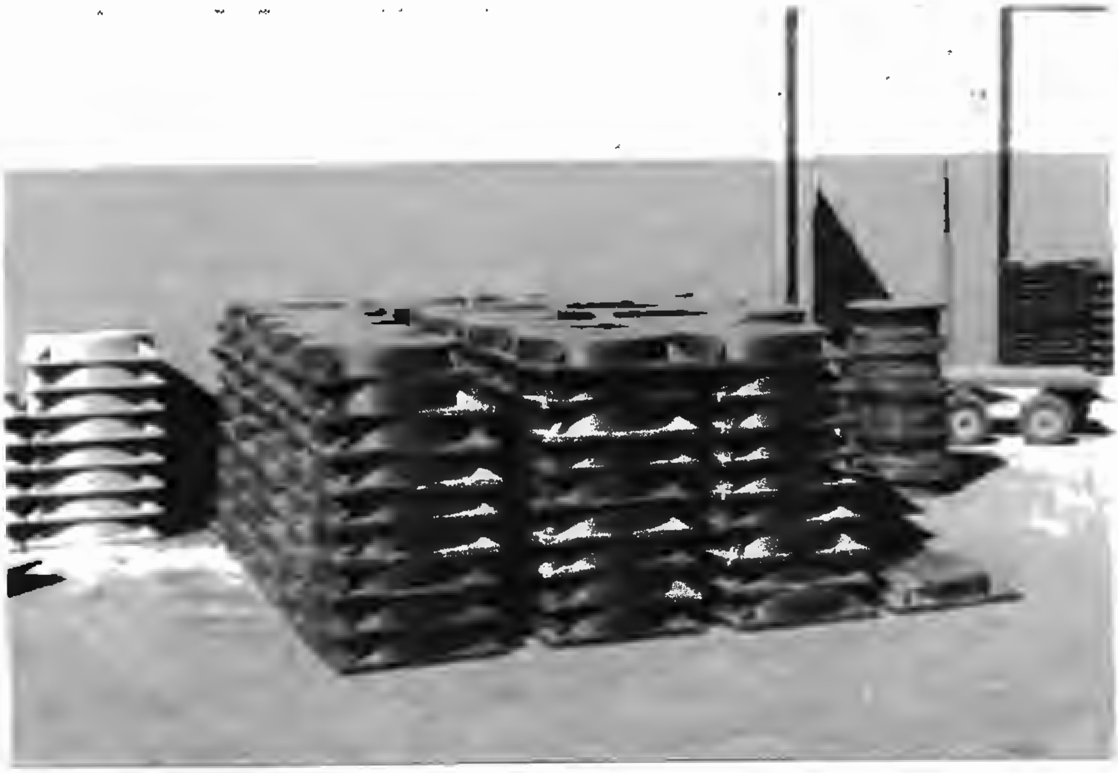

Figure 1.1. Stack of modified Type 2A polymer concrete manhole covers and frames at the AV Mouldings factory. Standard Type 2A replacement covers are visible behind the stack to the right.

2. LITERATURE REVIEW

2.1 Polymer concrete properties and applications

The generic term "polymer concrete" refers to materials combining a mineral aggregate with a resinous binding system. The aggregate may be coarse or fine, or consist of a blend of particle sizes. The resin used may be one of various types, including epoxy, polyester, vinyl ester, formaldehyde and methyl methacrylate. These resins all form part of the large group of polymeric materials called thermosets. They are so called due to their behaviour at high temperatures. Unlike thermoplastic materials, they do not have a melting point, and thus (to a large degree) retain their mechanical properties at high temperatures. However these materials do undergo chemical breakdown, and can become a fire hazard under such conditions.

Polymer concrete can be made by mixing resin with a suitable aggregate in convenient proportions. The percentage resin can vary from as little as 10 wt % up to about 25-30 wt %. Rao and Krishnamoorthy[1] have found evidence to suggest that there is no benefit in making polymer concrete with high resin contents. In theory, only enough resin is needed to coat all aggregate particles, and to fill all voids between them. It is desirable to reduce the resin content as much as possible, since it is the costliest component of the polymer concrete mix. Another factor that should be considered is the workability of the wet mix. Polymer concrete with low resin contents can become difficult to work with, and this affects its mouldability.

Chopped strand fibre reinforcement is sometimes used to provide extra strength to polymer concrete intended for load bearing applications. These glass fibres are usually supplied in the form of little bundles held together by a binding agent. When mixed into a resin, shear action in the mixer causes these fibres to dissociate from their bundles into the resin.

The mechanical properties of polymer concrete in general are superior to those of ordinary concrete. Strength in tension, compression, and flexure are 3 to 5 times higher, and it is more tolerant of shock loading. Its wear properties are similar to those of ordinary concrete, and its resistance to aggressive environments is (in most cases) acceptable. The cost of polymer concrete is at least 3 times higher than ordinary concrete, however, and this is one of the main reasons why it is not more popular for engineering purposes.

Browne, Adams and French[12] have compiled a fairly comprehensive summary of the advantages and disadvantages of polymer concrete. Some of the more relevant facts revealed in their analysis are now discussed.

In general, resin concretes have strengths that vary between 14 and 140 MPa in compression. The resin acts as the binder between the filler particles, implying that the strength of the polymer concrete is dependent on the strength of the bond between the resin and the filler. An aggregate system graded to give a minimum void content should be used. In addition, as the resin content is reduced below that required to fill all voids, the strength falls off very quickly.

High strength polymer concrete can be made by using an aggregate graded to give a high packing density. Compressive strengths as high as 140 MPa have been reported using mixes designed this way. However, such mixes are not very workable and need to be extremely well compacted to achieve these results. Flexural and tensile strengths are limited by the bond strength between the resin and the aggregates. Angularities in aggregate shape are also known to cause stress concentrations in the resin matrix. Consequently the flexural and tensile strengths of polymer concrete are typically only about 30 % of it's compressive strength.

The effect of moisture is detrimental to the mechanical properties of polymer concrete. Vipulanandan and Mebarkia[2] established that polyester polymer specimens are not significantly affected by water degradation, while polyester polymer concrete specimens are adversely affected. This implies that moisture attacks the resin-filler interface, rather than the resin itself. Browne, Adams, and French[12] have verified this theory. Even with a well filled polymer concrete system, they observed a 45 % drop in strength after 2 weeks submersion in water, whereas the polyester resin specimens did not exhibit this behaviour.

The creep behaviour of polymer concrete has been studied by Taylor Woodrow Laboratories. Polyester concrete specimens and ordinary cement concrete specimens were loaded to a stress/strength ratio of 17 %. After 100 days at 20 °C, the polyester concrete had a strain nine times higher than the cement concrete. For this reason, polyester polymer concrete is not recommended for use in cases of sustained high loading at elevated temperatures.

The impact resistance of polymer concrete is generally greater than that of ordinary concrete, owing to its much higher strain capacity. Polymer concrete specimens withstood impact loads more than twice those of ordinary concrete in a test developed by Taylor Woodrow Laboratories.

One factor that has been known for years is that individual test results involving polymer concrete (particularly fibre reinforced polymer concrete) are often subject to a significant amount of statistical variation. For this reason *at least* five to ten specimens are usually tested before any analysis is undertaken.

Polymer concrete has traditionally been used mainly for non-load bearing applications. Examples of these would be industrial flooring (polymer concrete has excellent wear resistance), piping, ducting, decoration, aesthetic mouldings, and also repair of damaged concrete structures. Some examples of load bearing applications that polymer concrete is becoming more popular for are machine foundations, pump and gear cases (due to its good sound and vibration damping properties), as well as manhole covers and frames.

2.2 Raw Materials

2.2.1 Resins

The resins used in most polymer concrete nowadays are the unsaturated polyesters and the epoxies. Other resins are occasionally used, but it is generally accepted that the properties (including cost) of polymer concrete made from these resins are the most desirable. Epoxy resins are about 2 – 3 times more expensive than the polyesters, but are used where good toughness, low shrinkage during curing, and good resistance to alkalis is required. The unsaturated polyesters are used extensively for laminating and moulding. They are usually diluted with a liquid vinyl monomer, such as styrene, during manufacture. This styrene plays an active part in the curing of the resin. Since the focus of the research conducted during this project involves polyester resins, no further mention will be made of the other generic types of resin.

Typical properties of a cured polyester resin without filler or reinforcement are given in the following table (extract from Crystic Polyester Handbook[14]): -

Specific gravity	1.28
Hardness, Rockwell M scale	110
Barcol hardness*	45
Tensile strength	70 MPa
Compression strength	140 MPa
Impact strength, notched Izod	0.54 J
Tensile modulus	3.5 GPa
Specific heat	2.3 kJ/kg K
Thermal conductivity	0.2 W/m K
Coefficient of linear expansion	100 X 10^{-6}/°C
Water absorption 24h at 20 °C	0.2 %

Table 2.2.1. Typical properties of cured polyester resin.

Polyester resins are available in two main types. These are the orthophthalic, and the isophthalic unsaturated polyesters. Isopthalic polyesters are known to exhibit better mechanical properties, but are also more expensive than the orthophthalic polyesters. Recently, a new type of resin has become available. This is a resin based on a dicyclopentadiene (DCPD) system. However, the current school of thought is that these resins are too brittle to be used for polymer concrete.

Polyester resins require the addition of an accelerator and a catalyst to start the curing reaction. Variation in the amounts of these chemicals added to the resin affects the rate of cure, gel time, and the degree of cure. The curing reaction is exothermic, and

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* Barcol hardness is a common method of expressing the hardness of a cured resin. It is derived from the depth of an indent produced from a standardised indenter.

the heat evolved can be considerable. Polyester resins are available in pre-accelerated form ie. they require only the addition of the catalyst to start the curing reaction, as well as thixotropic, and low viscosity varieties. Each of these properties of the resin has advantages when applied to polymer concrete, eg. thixotropic resins reduce segregation of aggregates during moulding; low viscosity resins (in theory) allow a lower overall resin content, whilst retaining a suitable mix workability.

2.2.2 Aggregates

The aggregates (or fillers) commonly used for the manufacture of polymer concrete vary widely. They can generally be divided into two groups, being fine aggregate and coarse aggregate. Examples of fine aggregate are river sand, dune sand, fine quartz, calcium carbonate, silica flour, and fly ash. An example of coarse aggregate is crushed stone (traprock, limestone, gravel, quartzite, shale etc.). For the purposes of polymer concrete, calcium carbonate, silica flour, and fly ash are classified as microfillers. These fillers consist of particles with sizes of the order of microns. Most researchers agree that particles with sizes of the order of 2 mm and upward may be considered to be coarse.

It has been shown during previous research by Rao and Krishnamoorthy[1] that polymer concrete of higher strength can be made by using a graded aggregate mixture or blend of aggregate sizes, than is possible using one size of aggregate. The reason for this is that the strength of polymer concrete is related to the void content of the hardened material. A graded aggregate blend can provide the optimum filler content, thus reducing the number of voids, as well as reducing the required resin content of the mix. However, as mentioned before, the workability of the wet mix must be taken into account, when deciding on an aggregate system.

All aggregates should be completely washed and dried, and free from foreign organic material (or any other contaminants). Particle shape is also important in determining the properties of the hardened material. Rounded particles are preferred to angular particles. Randomly angular particles often do not fit together in the hardened material as well as rounded particles, and sharp corners can cause the hardened resin binder to crack at these points, due to setting shrinkage stress, or indeed any other applied stress.

Figure 2.2.1 illustrates an idealised situation for large *rounded* particles in hexagonal close packed structure incorporating two sizes of microfiller particles.

It should be noticed that there is a theoretical maximum filler content that can be attained with this packing arrangement. There will always be gaps in the aggregate structure no matter how small the filler particles are. The volume of the large aggregates is about 80 % of the total volume of the material. The volume of the intermediate size particles is about 6 %, and the volume of the small particles is about 2.5 % of the total volume. As smaller particles are added, their contribution to the total volume tends towards zero. In practice it seems that a total filler content of about 90 % may be attained with careful aggregate grading methods. In addition, the kind of idealised packing illustrated is difficult to achieve in practice

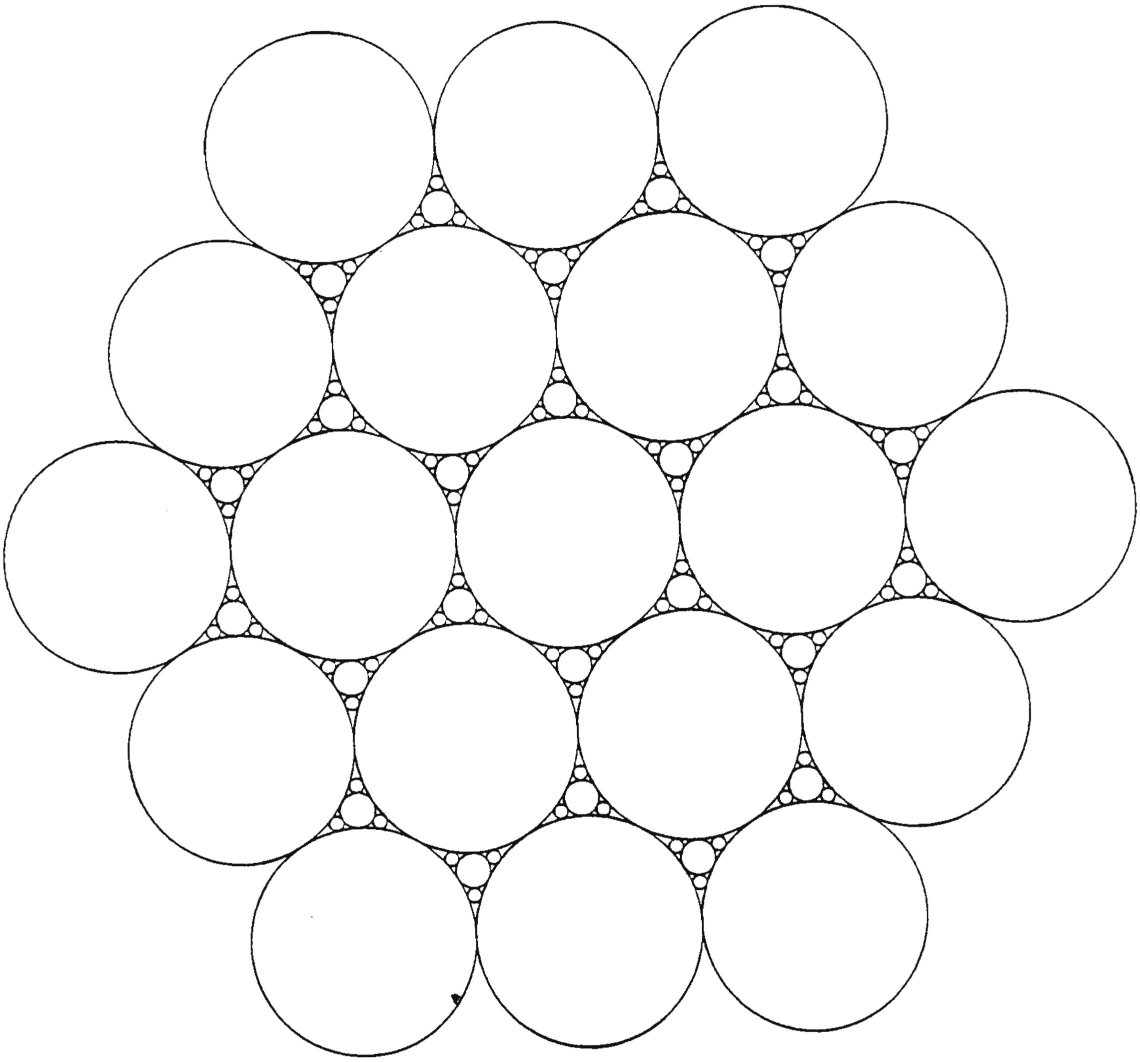


Figure 2.2.1 Idealised packing of rounded particles in two dimensions.

According to Capuano[5], polymer concrete of varying densities can be produced using aggregates of different weights. For example, lightweight materials are made using synthetic fillers such as hollow glass or ceramic beads. Heavier materials are made using heavy aggregates such as baryte. In many cases, fillers are used in polymer concrete only to add weight, bulk, and rigidity to the material. Their direct influence on the strength of the material is not always a positive one. In most cases of failure of polymer concrete products, brittle fracture resulting from crack propagation is the primary cause.

2.2.3 Pigments

The idea that polymer concrete can be pigmented to produce almost any desired colour is attractive from an aesthetic point of view. Concentrated pigment paste can be added to the mix in small amounts to achieve the required colour. Pigments have a negligible effect on the properties of polymer concrete. However, the effect of UV radiation absorption by certain colours has been considered. It is known that UV radiation can have a detrimental effect on polymeric materials, but as yet no reliable information is available to quantify the effect on polymer concrete.

2.2.4 Chopped strand fibres

Glass fibre reinforcement of polymer concrete is common practice. In the same way that fibres are added to plain resin structures (eg. laminates), they can be added to a polymer concrete mix. Glass fibres are usually supplied in bundles held together by a binding agent, and chopped at certain lengths. On mixing, the fibre bundles dissociate into the resin in random orientations. Research by Valore and Naus[8] showed that the addition of fibre reinforcement results in an increase in the mechanical strength of polymer concrete. In addition, their work showed that higher flexibility is possible. Longer fibres appeared to be more effective, and higher modulus fibres produced better results. The disadvantage of adding fibres to the resin is that they increase the viscosity of the mix considerably, with the result that a higher resin content is required to achieve an acceptable workability. This in turn increases the cost of the material.

Some of the results of Valore and Naus's work on fibre reinforcement in polymer concrete are presented in figure 2.2.2.

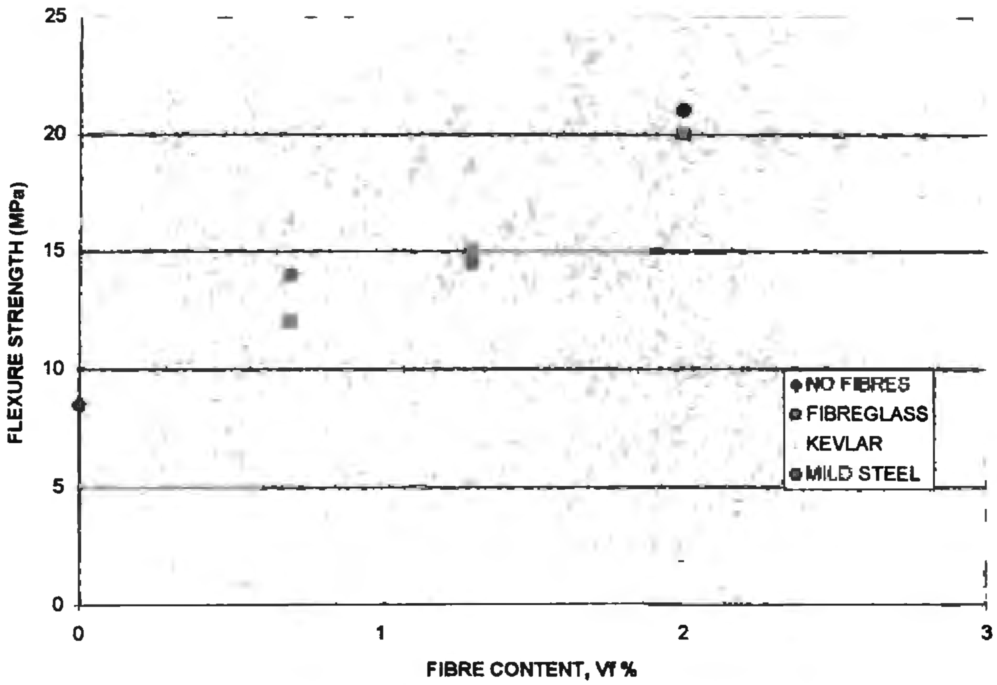


Figure 2.2.2. Effects of different types of fibre reinforcement in polymer concrete, according to Valore and Naus[8].

2.2.5 Woven roving reinforcement

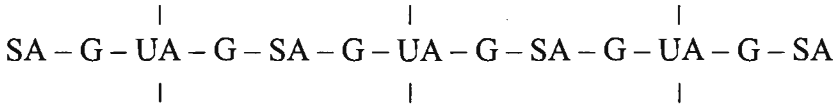
Very little published work is available regarding the reinforcement of polymer concrete with woven roving. This type of reinforcement consists of glass fibre mat that can be placed at certain positions in a polymer concrete moulding to provide extra strength. In some cases, woven roving reinforcement can be detrimental to the mechanical properties of polymer concrete mouldings. This is because it is often difficult to achieve sufficient impregnation of the mat by the resin, particularly in polymer concrete with low resin contents. The woven roving then becomes a weak point in the material, and fracture of the product can occur along the plane of the reinforcement. However, it is known that careful design of the mix, along with sensible placement of the woven roving in the mould, generally strengthens the polymer concrete product.

2.3 The Resinous Binding System

2.3.1 The curing reaction

Thermosetting resins generally become increasingly infusible on heating. This process is called polymerisation or curing. Some resins (phenol formaldehyde, urea formaldehyde, and melamine formaldehyde) cure through a condensation reaction during which water or steam may be evolved. Polyester resins are also thermosetting liquids, but they cure through a true polymerisation reaction during which no volatile by-products are formed.

Unsaturated polyester resins are derived from coal and petroleum products, the basic building blocks being the saturated dibasic acids, unsaturated dibasic acids, and glycols. Commonly used saturated and unsaturated acids are the phthalic anhydrides, and the maleic anhydrides. The basic structure of the resulting condensation product can be represented as: -



$$\mathrm{SA-G-\underset{|}{\overset{|}{UA}}-G-SA-G-\underset{|}{\overset{|}{UA}}-G-SA-G-\underset{|}{\overset{|}{UA}}-G-SA}$$

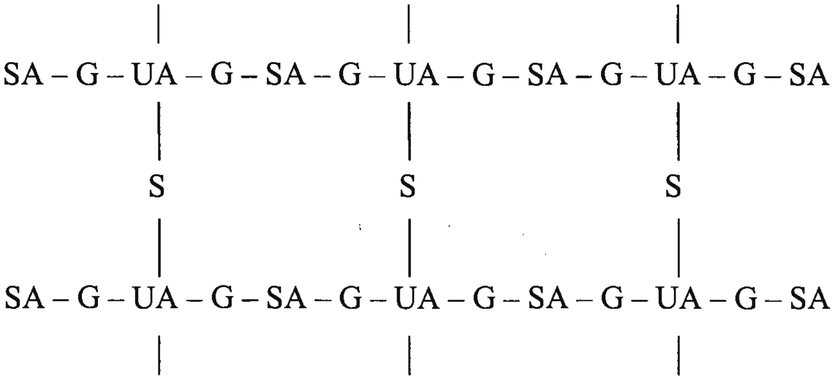
where SA is the saturated acid

UA is the unsaturated acid

G is the glycol

This structure is called an unsaturated polyester base. The unsaturated acid groups in the above chain are so called because they are capable of linking up with other monomers. They cannot link up with themselves due to the shape and size of their molecules. However, if another unsaturated polymerisable compound is added, this can interact with the groupings between linear molecular chains and effectively crosslink these chains into complex three-dimensional networks. This compound is known as the crosslinking monomer, and is in most cases monomeric styrene. Thus, polyester resins consist of an unsaturated polyester base diluted in monomeric styrene. The amount of styrene can also control the viscosity of the liquid resin.

The crosslinked polyester resin structure is shown: -



where S is the crosslinking monomer

The above description of a polyester resin implies that left to itself, it would eventually crosslink itself into what is effectively one giant molecule. This is the case, and as a consequence all resins do have a useful storage life. The influence of heat and light can accelerate the crosslinking process, and thus reduce the storage life. For this reason, polyester resins are kept in sealed metal drums, in a cool dry place.

For practical purposes the crosslinking reaction is much too slow to be useful. It can be accelerated by the use of catalysts (also called hardeners or initiators) which decompose into free radicals, effectively increasing the reactivity of the polymerisation process in the polyester resin. Accelerators (also called promotors) are also added to speed up the decomposition of catalysts. They are used mainly in cold curing applications, since the rate of decomposition of the catalyst is sufficiently fast at higher temperatures. The most commonly used catalysts are organic peroxides such as methyl ethyl ketone peroxide (MEKP). Accelerators are often amines, or cobalt soaps such as cobalt naphthenoate and cobalt octenoate, and they are used to control the setting time of the resin. The amount of catalyst added also affects the setting time of the resin, but it is reportedly not good practice to vary the amount of catalyst, since too little can lead to undercure, while too much can result in extreme exothermic reactions.

Some polyesters are supplied in pre-accelerated form, and require only the addition of the correct amount of catalyst to start the curing reaction. The gel time of a typical pre-accelerated polyester resin is usually about 20-30 minutes.

The following factors affect the gel time of a polyester resin :-

- Catalyst content.
- Accelerator content.
- Ambient temperature. Higher temperatures speed up the curing rate.
- Bulk of resin. Larger bulks of resin tend to set faster than smaller ones.
- Loss of crosslinking monomer by evaporation. Styrene is volatile and evaporates over time. Thus there may be insufficient monomer to completely cross-link the polyester chains.
- Addition of fillers. Higher filler contents increase the setting time. The presence of moisture retards the polymerisation process.
- Pigmentation. Some pigments increase the setting time, while others tend to decrease the setting time.
- Presence of inhibitors. Certain contaminants such as phenols, sulphur, copper salts, carbon black, and methanol may inhibit the polymerisation process, slow down the curing rate, and in some cases prevent full cure altogether.

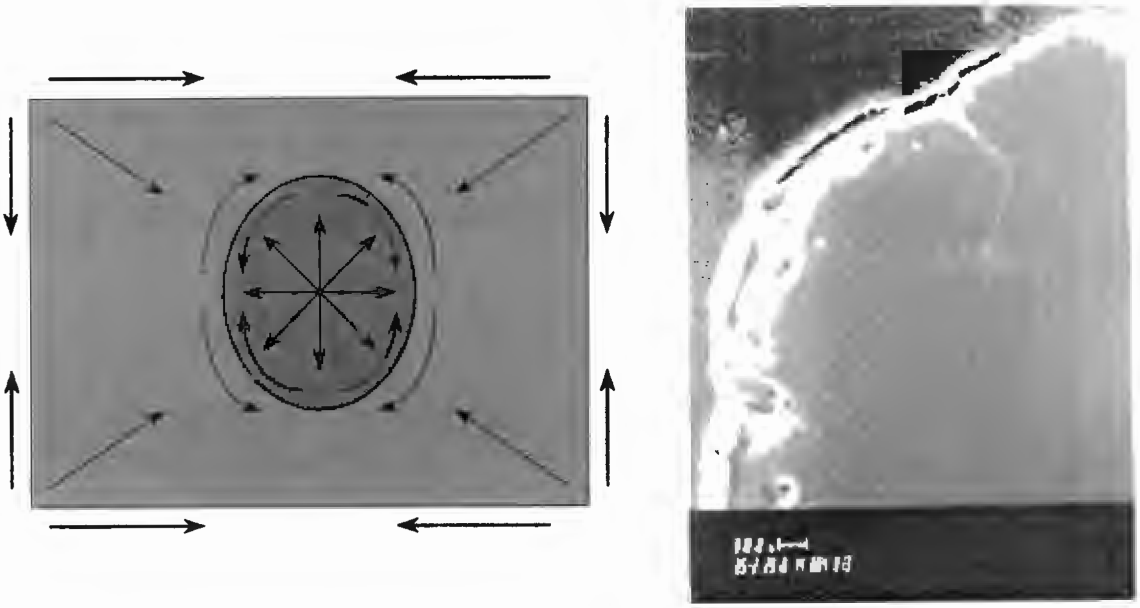
In addition to these factors, improper mixing of the catalyst (or accelerator) into the resin can result in 'wet spots' in the finished moulding.

Post curing of finished products can be performed to ensure complete cure. This process is normally performed after allowing the moulding to cure at room temperature for 48 hours. Post curing is carried out by exposing the moulding to elevated temperatures for a relatively short time. Methods differ from resin to resin, but typical post curing treatments are performed at 80° C for 3 hours. Complete cure of the mouldings is desirable, since this results in the optimum mechanical properties of the product being achieved.

2.3.2 Defects in polymer concrete mouldings

Poor resin-aggregate adhesion

The hardening of a polymer concrete mix results in the aggregates effectively becoming embedded in the resin matrix. Various considerations have to be made to evaluate the compatibility of the two materials with each other. One factor that should be noted is that there is a certain amount of setting shrinkage involved with the resin curing reaction. At the resin – sand particle interface this can result in surface stresses being introduced. It is clear that shrinking resin puts the surface of the sand particles into compression, while the reaction (resistance) of the sand particle to this effect puts the resin surface at the interface into tension. This results in a significant amount of internal stress that can be developed in the hardened material. The effect is illustrated by the following diagrams: -



(a) *(b)*

Figure 2.3.1 (a) Diagram illustrating the internal stresses that can be developed due to setting shrinkage (curved arrows indicate the direction of the surface forces, not their positions), (b) SEM photograph showing poor resin-sand particle adhesion[24].

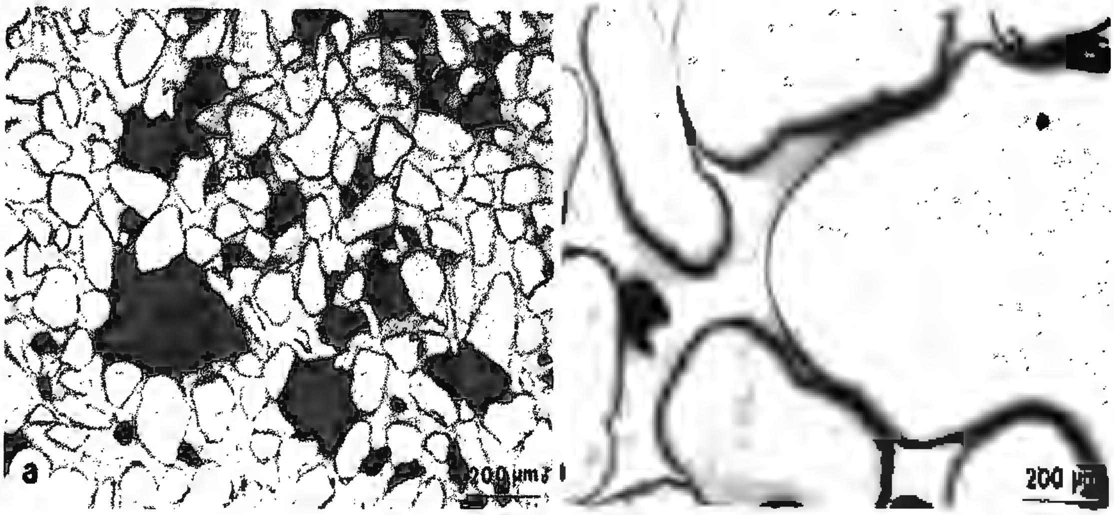
The arrows in figure 2.3.1 (a) indicate the direction of the shrinkage force on an aggregate particle in a resin matrix during curing. The resin surface at the interface experiences tensile forces.

The problem illustrated in figure 2.3.1 (b) may also arise due to inadequate mixing of the resin and aggregates. If all aggregate surfaces are not completely wetted by the resin, then it is likely that there will be poor resin-aggregate adhesion in the finished moulding.

It should be clear that defects such as the one just discussed, are not desirable in a polymer concrete moulding. They provide perfect paths along which cracks can propagate, and thus render the moulding susceptible to fracture.

Voids

It is well known that the mechanical strength of polymer concrete is inversely proportional to its void content. For this reason, effort is usually made to minimise the number of voids in a polymer concrete moulding, eg. by vibratory methods. Voids are the result of air bubbles in the moulding. These air bubbles are stirred into the resin during the mixing process, and are unable to escape due to the viscosity of the resin. The problem can be minimised by proper *compaction* of the mix in the moulds, but it should be obvious that *stirring* of the mix in the mould is not beneficial. Indeed, this procedure may introduce more air bubbles into the mix. Work by Joneja and Newaz[24] focusing on the effect of fine and coarse aggregate revealed that voids in polymer concrete are a major problem in polymer concrete.



(a) *(b)*

Figure 2.3.2 (a) and (b). Photographs showing voids in polymer concrete[24]. Particles in (a) are fine aggregate, particles in (b) are coarse aggregate.

Fibre reinforcement

The purpose of introducing fibre reinforcement into polymer concrete mouldings is to strengthen the material, and to improve its fracture toughness. However, it is possible that chopped strand fibre additions can have the effect of providing uneven strength distributions in mouldings. As the fibres are mixed into the resin, they dissociate from their bundles by shear action in the mixer. In this form, they increase the viscosity of the resin substantially. They also tend to coalesce into "clumps" or form fine matting, similar to a woven roving mat. The effect is most noticeable if the resin and fibres are mixed for too long. If polymer concrete mix with these types of fibres is made into a moulding and set, then it is likely that the final product will display properties that are not homogenous throughout.

This kind of defect is not necessarily detrimental to the mechanical properties of the product. However, it does make the determination of a product's strength substantially more difficult, and introduces a significant statistical strength distribution from product to product.

Wet spots

Wet spots are the result of inadequate mixing of catalyst or accelerator into the polymer concrete mix, and the effect is that curing of the resin is retarded in certain regions of the moulding. Wet spots can appear on the surface of the moulding, but a far more dangerous scenario is one where wet spots form internally, and are thus not detectable by visual inspection methods.

The effect on the mechanical strength of the final product is that it is weakened, and becomes susceptible to fracture by crack propagation.

Miscellaneous defects

Other forms of defects include the presence of foreign particles in the mix, such as organic material, outsized aggregate particles etc.

All of these defects render products produced with these mixes susceptible to failure by fracture.

2.3.3 Effect of silane coupling agents

Coupling agents are sometimes used in resin – glass laminates to improve the adhesion between these two components. Glass fibres are treated with coupling agents during manufacture. These agents are often based on chrome complexes or silane compounds, but new and improved types of coupling agents are constantly being developed. According to previous research by Scott Bader Laboratories[14], coupling agents also render the surface of the glass fibres less hydrophilic, improving curing under humid conditions.

It has been reported that coupling agents can be beneficial to the mechanical strength of polymer concrete. They improve the adhesion between resin and glass fibres, as well as adhesion between resin and fillers (aggregates).

Mebarkia and Vipulanandan[2] found that the addition of silane coupling agents offset the decrease in strength associated with water immersion (see section 2.4). They experimented with polyester polymer concrete specimens under silane treated and untreated conditions. For the case of the silane treated specimens, the silane was mixed with the aggregates prior to addition to the resin. Figure 2.3.3 presents the results of the experiments. The untreated polymer concrete and the silane treated polymer concrete were immersed in water for 1 month. The polyester polymer was immersed for 3 years. The silane treated polymer concrete not only exhibited higher strengths than untreated polymer concrete, but the drop in strength due to water immersion was also reduced from about 21 % to about 7.5 %. The fact that pure polyester resin only marginally loses strength after even 3 years immersion in water confirms that the loss in strength must be due to attack of the resin aggregate interface in polymer concrete.

Kobayashi and Ito[10] conducted compressive fatigue tests on polyester polymer concrete specimens involving silane coupling agents. Their tests revealed that fatigue strength is improved from about 59 % to about 64 % of the static strength of the material when 1 % silane (by weight of resin) is added to the material. The silane agent used was γ-methacryloxyprophyltrimethoxysilane. The effect of silane additions on the fatigue life of polymer concrete is illustrated in figure 2.3.4.

The results of another investigation by Vipulanandan and Mebarkia[22] confirmed the fact that coupling agents improve the mechanical strength of polymer concrete. Their work involved flexural testing of polymer concrete specimens containing 6 % chopped strand fibres (by weight of resin), and incorporating silane treated aggregates. In addition, the aggregates were well graded (0.1 mm – 5 mm size). They found that the flexural strength of these specimens was 95 % higher than that of specimens without chopped strand fibre *and* silane additions. However it was noted during testing that the modulus of the material remained almost unchanged by these additions.

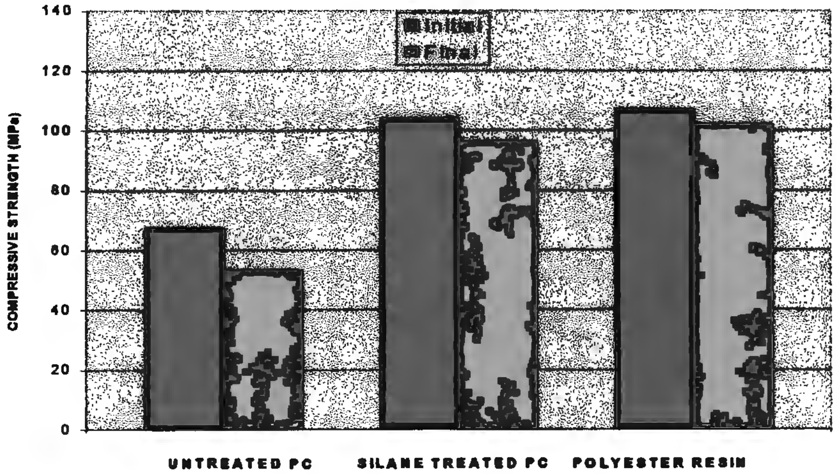


Figure 2.3.3. The effect of silane on strength loss due to water immersion[2].

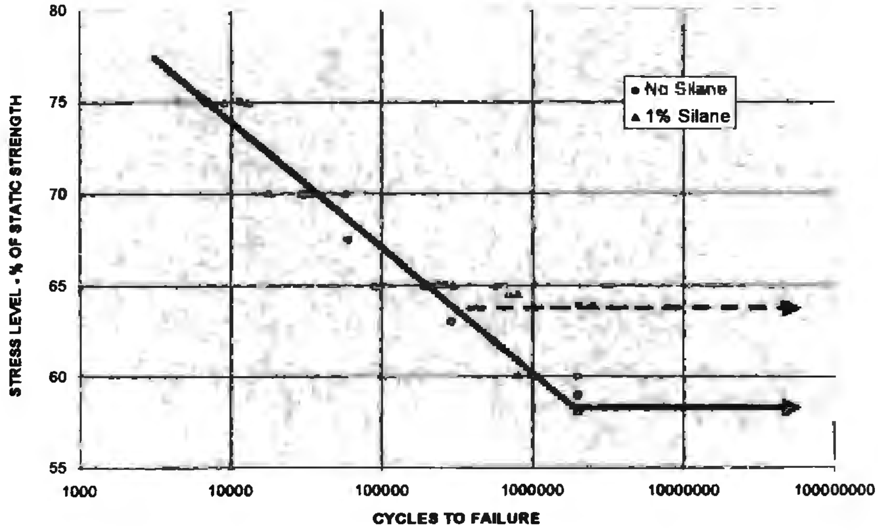


Figure 2.3.4. Effect of silane on the fatigue life of polymer concrete[10].

2.4 Degradation of polymer concrete

Polymer concrete is susceptible to various forms of degradation in service. This section is presented to describe the types of degradation possible.

2.4.1 Degradation by aggressive environments

Mebarkia and Vipulanandan[2] have completed extensive research into the effect that acids, alkalis, and other aggressive environments have on the mechanical properties of polymer concrete (without fibre reinforcement). They exposed polyester polymer concrete to sulphuric acid, sodium hydroxide solution, water, and sodium chloride solution for varying lengths of time. They found that all of these environments reduced the compressive and tensile strengths of polymer concrete. The amount of the decrease was found to be dependent on the type, concentration, and pH of the solutions used. Their study also analysed the effect of liquid uptake (absorption) on polymer concrete. They measured weight changes in specimens, as well as drying times, and residual weight increases after drying.

It was found that alkaline solutions with a pH of 13 or higher produced the most significant decrease in compressive strength of polyester polymer concrete. The figure below illustrates this deterioration after one month immersion. For the NaOH solution the loss of compressive strength was about 28 %.

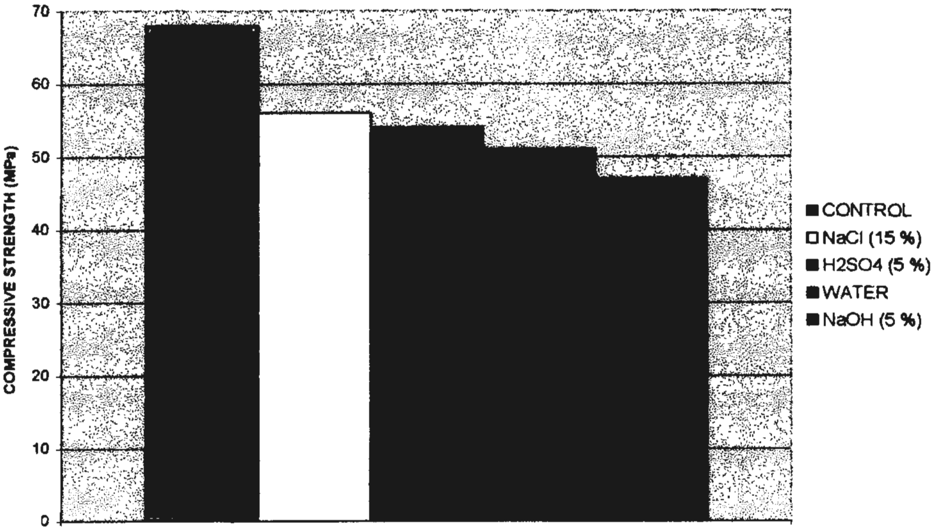


Figure 2.4.1 Compressive strength reduction after 1 month in various corrosive solutions[2].

2.4.2 Degradation by solvents

The effect of solvents on the mechanical strength of polyester polymer concrete has not been well researched. Browne, Adams, and French[12] assessed the durabilities of polyester polymer concrete based on tests conducted by Taylor Woodrow Construction Laboratories. Test specimens were exposed to various chemicals for a period of three years. These chemicals included various oils, fats, as well as diesel fuel. It was found that there was a certain amount of visual degradation, eg. yellowing, but the tests did not include mechanical strength measurements.

It is not well known how polyester polymer concrete behaves after exposure to solvents such as ethanol, methanol, gasoline, diesel fuel, engine oils etc. However, it is suspected that these types of chemicals can have a detrimental effect on the mechanical properties of polyester polymer concrete.

2.4.3 Degradation by UV radiation

It is well known that polymeric materials are susceptible to degradation by ultra violet radiation. Radiation of wavelength shorter than 360 nm is capable of breaking C-C bonds in polymeric materials[13]. Also, short wave radiation (around 300 nm) can result in photo oxidation of groupings on the polymer chains. The mechanism of degradation is not entirely understood, but it is thought to be the result of these two effects combined. For this reason, it is desirable to quantify the extent to which degradation is possible. No previous research regarding UV degradation of polyester polymer concrete could be consulted*.

The purpose for which polymer concrete mouldings are produced determines how significant the problem may be. For example, a polymer concrete floor inside an industrial warehouse would not be exposed to significant amounts of UV radiation by the sun. However, a polymer concrete manhole cover in a road may be exposed to UV radiation for a prolonged period. UV radiation exposure on polymeric materials generally produces an embrittling effect. This may not affect the *strength* of products initially, but would certainly decrease the fracture toughness of the material, and render it more susceptible to fracture under cyclic loading conditions.

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* Scott Bader[14] reports that 'yellowing' of glass fibre reinforced polyester laminates can occur due to UV radiation. Apparently this does not affect the mechanical properties of mouldings.

2.5 Polymer concrete for manhole covers and frames

AV Mouldings (Pty) Ltd. currently manufacture manhole and drain components from polymer concrete. Their products are based on specifications set out in SABS 558:1973, and SABS 1115:1976 (thickness dimensions vary). These specifications are set out for products manufactured from grey cast iron, but since no specification exists for polymer concrete products, they adhere to the above mentioned specifications. Whether or not these specifications are applicable to polymer concrete products is considered to be one of the main objectives of the research undertaken in this project.

2.5.1 The manufacturing process

AV Mouldings currently use polymer concrete characterised by the addition of mainly fine aggregates. The mixes made up on their premises are not varied substantially, apart from the occasional employment of different resin suppliers. The following table summarises the ingredients and proportions currently used: -

Component	Proportion (wt %)	Type	Supplier
Resin	21	Pre-accelerated orthophthlalic polyester	Harvey's Fibreglass Scott Bader
Aggregate	78	No. 1 Grade washed/dried (0.1 – 1 mm)	Consol Industrial Minerals
Pigments	0.2	Various grades and colours	Pyramid Chemicals
Fibres	0.5	12.5 and 18 mm (50/50) chopped strand fibres	Owen's Corning
Matting	1 or 2 layers	Woven roving matting	Owen's Corning

Table 2.5.1. Mix design currently used by AV Mouldings.

The manufacture of polymer concrete involves the mixing of the resin with the chopped strand fibres, allowing them to dissociate out of their bundles. The sand aggregate is then added in the correct proportion. Finally pigment is added to achieve the required colour for the particular mix. The mix is then poured into drums for storage. Required amounts of mix are then taken in buckets to the moulding area, where MEKP catalyst is added in the required amount (2 – 4 %), depending on the ambient temperature). After thorough mixing in of the catalyst, the mix is placed in the moulds by hand, and evenly spread. For the case of certain heavy-duty products, woven roving reinforcement is placed through the thickness of the moulding at various intervals. Thorough compaction of this matting by hand is usually enough to achieve sufficient impregnation (print through) of the matting by the resin.

The moulding is left to stand for about 30 minutes, after which time it is normally sufficiently hardened to be removed from the mould. On removal, any chips or irregularities are scraped off the surface. The product is then left to cure at room temperature for at least 24 hours, before final surface finishing is carried out. All products are weighed to ensure that they comply with minimum weight requirements.

All buckets and contact implements (eg. scrapers) are cleaned out, and the weight of any wastage is recorded (for the purpose of optimising efficiency). Skin irritation from contact with wet polymer concrete mix, can cause severe discomfort, thus it is common practise to wear gloves or use barrier creams when handling wet mixes.

Sample products are occasionally selected for testing on an SABS calibrated testing bench. These products are tested according to the method described in SABS 558:1973[23]. This test involves the following procedure: -

1) Covers are tested in their respective frames.
2) A uniform layer of plaster of paris is cast in the centre of the top of the sample.
3) The sample is positioned on a resilient material in the testing machine.
4) A steel or hard wooden block is positioned on top of the sample (on the plaster of paris layer).
5) The appropriate compressive force is applied to the top of the block at a rate not exceeding 36 kN/min. The forces to be used are 135 kN, 40 kN, and 7 kN for heavy, medium, and light duty classes respectively.
6) The force is maintained for at least 30 s. The sample is unloaded and inspected.

It should be noted that this testing method is a *static* test. The load is applied slowly in order to allow strain to develop. This is often called equilibrium loading. It should be clear that a manhole cover in service would not necessarily be subjected to this type of loading. It is more likely that a manhole cover in service would be subjected to high cycle impulse loading. In other words, the failure of manhole covers is probably brought about by brittle cracking, rather than a strength deficiency. However, there is a relation between the static strength and the fatigue strength of these covers, and the SABS testing procedure employs a large factor of safety. It is unlikely under normal conditions that a manhole cover in service would be subjected to a direct force higher than 80 kN. Further discussion of the loading condition of manhole cover is given in section 2.6, and a full stress analysis is presented in section 6.

2.5.2 Product compliance with standards

It has been found that polymer concrete manhole covers and frames comply with SABS testing requirements set out in SABS 558:1973[23] and SABS 1115:1976[24]. It should be remembered however, that these specifications describe the testing procedure for cast iron covers. Whether they may be considered relevant to polymer concrete covers is an issue to be investigated.

AV Mouldings products in general have performed well in service. A significant part of their product range, however, are intended for non-load bearing (or light duty) applications. At present only one of their products has been known to fail in service. This is the Type 2A heavy duty replacement cover, common on many South African roads.

2.6 Methods of stress analysis of manhole covers

This section is included here to describe the loading condition and the equations used for stress analysis of manhole covers in service. The term 'manhole cover' generally refers to any type of grating or cover that fits into a road or walkway to cover an access opening to storm water or sewer systems. The equations describing the applied stress on covers vary considerably according to the shape and size of the cover.

2.6.1 Loading condition

It is the author's view that there are two main types of loading that can occur. These are static loading and dynamic loading. It should be noted that these two categories include every conceivable loading configuration possible eg. tensile, compressive, flexural, torsional etc. The two categories merely describe the nature of the *application* of the load (in any configuration). Dynamic loads may further be divided into two categories, namely suddenly applied and impact loads. Suddenly applied loads are often called impulse loads, and involve the sudden appearance of a load. Impact loads involve the collision of two bodies (shock loading). If one considers the nature of the load induced when a vehicle passes over a manhole cover, it should be clear that the loading condition does not involve the collision of two bodies, nor is it a case of gradually applied static loading. It is thus a case of impulse or suddenly applied loading.

Furthermore, the load itself is not a point (concentrated) load. It is spread out over a certain area over the cover, and this area may vary in size and position as a vehicle passes over the cover. From this preliminary analysis it should be clear that the loading condition on a manhole cover in service, is not a simple issue. The magnitude, rate of application, and size of distributed area of the load can vary considerably.

For this reason the analysis undertaken here and further on in this report always assumes the worst case scenario. This is because the main interests here are the maximum strength, toughness, and durability of the product.

2.6.2 Plate theory

The stress analysis of plates is considerably more involved than that of beams and other structural components. Nevertheless it has been extensively researched and is thus well understood. The equations used during the course of the research undertaken here are based on explanatory literature by Lowe[17], Ugural[18], Seely[19], and Griffel[22]. A plate subjected to a normal load, bends in all planes normal to the plate, whereas a beam may be considered to bend in one plane only. In addition, bending of a plate in one plane is influenced by bending in all other planes. Plate theory predicts maximum stresses that err on the side of safety, ie. the load that a plate will resist in practice is usually greater than that predicted by theory.

A manhole cover, whether circular, square, or rectangular, may be considered to be a flat plate. There are three classes of plates namely, thick plates, medium-thick plates, and thin plates. In thick plates, shearing stresses become important and bending is not the dominant stress condition (the resistance of a thick plate to bending deformation induces non-negligible shearing stresses in the plate). In medium-thick plates, bending is the main action on which the useful resistance of the plate depends. In thin plates, the useful resistance of the plate depends in part on the direct tension accompanying the stretching of the middle plane.

Most manhole covers can be considered to fall into the class of medium-thick plates. The assumption in this case is that bending is the dominant action*. For this to be valid, it is required that the deflection of the plate be relatively small (less than half the thickness of the plate). If this is not the case, direct tensile stresses in addition to the bending stresses will contribute substantially to the load resistance of the plate.

It is not the intention of this report to delve into the derivations of the bending stress equations. They depend on various different configurations, such as whether the load is concentrated or distributed, and whether the edges are fixed or simply supported. For the case of manhole covers, it is generally assumed that the applied load will be distributed over some area, and that the edges are simply supported.

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* Although bending is the *dominant* action, there are still some shear effects that are in most cases negligible. As the thickness of the plate increases the shear force effects become more noticeable. For this reason the plate theory used here gives an *approximation* to the true loading resultants.

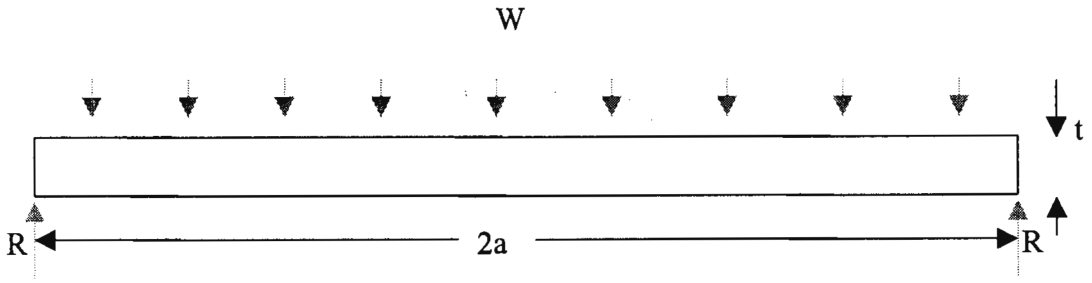


Figure 2.6.1 Circular plate under distributed loading condition.

For the loading condition shown, with edges simply supported, the bending stress as a function of radius r, is given by: -

$$\sigma = \frac{3Wz}{4t^3}(3+\nu)(a^2-r^2) \qquad (2.1)$$

where σ is the bending stress

W is the distributed load

t is the thickness of the plate

z is the thickness through the plate at which σ is to be determined

a is the radius of the plate

ν is Poisson's ratio

The maximum stress takes place at the centre of the plate on the surface (r = 0, z = t/2), and is given by :-

$$\sigma_{\max} = \frac{3}{8}(3+\nu)W\left(\frac{a^2}{\overset{2}{t}}\right) \qquad (2.2)$$

From these equations it is possible to determine the stress at any point on the surface of the circular plate (the maximum stresses will occur on the surface of the plate).

The deflections that a circular plate undergoes during the application of a load under the same conditions as for (2.1) may also be derived from plate theory. The deflection as a function of radius is given by equation 2.3, and the maximum deflection is given by equation 2.4.

$$\delta = \frac{Wa^4}{64D}\left[\frac{r^4}{a^4} - 2\left(\frac{3+\nu}{1+\nu}\right)\left(\frac{r^2}{a^2}\right) + \frac{5+\nu}{1+\nu}\right] \tag{2.3}$$

The maximum deflection is at the centre of the plate (r = 0): -

$$\delta_{\max} = \frac{Wa^4}{64D}\left(\frac{5+\nu}{1+\nu}\right) \tag{2.4}$$

where $D = \dfrac{Et^3}{12\left(1-\nu^2\right)}$ and E is Young's Modulus (2.5)

For the purposes of this investigation the stress analysis of Type 2A circular covers is of optimum concern, since these are the products that have been known to fail in service. However, rectangular and square covers are also manufactured by AV Mouldings, and thus bending stress equations for these products are also given.

The bending stress equation for rectangular plates is: -

$$\sigma = \frac{1}{2}\left[\frac{a^2}{\left(b^2 + a^2\right)}\right]\left(\frac{Wb^2}{t^2}\right) \tag{2.6}$$

where a is the length of the longer side

b is the length of the shorter side

For square plates, a = b, and equation 2.6 reduces to

$$\sigma = \frac{1}{4}\left(\frac{Wb^2}{t^2}\right) \tag{2.7}$$

These equations give the values of the average bending stress at the surface along a diagonal across the plate.

The fact that most of the manhole covers considered do not have a uniform thickness throughout introduces complications into the calculations of bending stress. Most of the products considered consist of a bulk section and a support lip around the edges. The diagram below illustrates a typical Type 2A replacement cover.

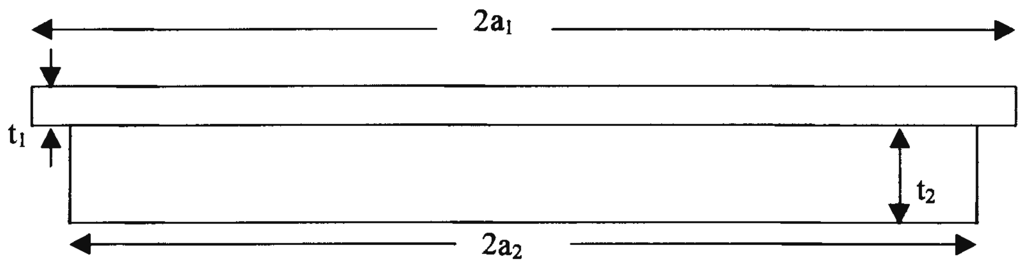


Figure 2.6.2. Side view of a typical Type 2A manhole cover

It can be seen that a manhole cover like this essentially consists of two plates ie. a lip section and a bulk section. The load applied to the cover is supported by the lip section. There are two main reasons for including the bulk section. One is that it provides weight to the cover, ensuring that it does not bounce out of the frame. The other is to provide rigidity to the lip section, ensuring that large deflections do not occur. Large deflections of the lip section can result in cracking through the middle of the cover. Nevertheless, the stress distribution in the section shown can become complicated to derive. It should also be noted that the small radius under the lip section can cause stress concentrations, which under certain conditions, may result in circumferential cracking in these regions.

The stress distribution found from equation 2.1 is normally considered to be an *approximation* that errs on the side of safety. Unlike many other loading configurations, the bending stresses and deflections in a circular plate are dependent on material properties (Young's modulus and Poisson's ratio). It is thought that E and ν for polymer concrete are likely to vary depending on the constituents of the mix.

Mantrala and Vipulanandan[4] conducted non destructive tests on polyester polymer concrete specimens using impact resonance methods to determine E and ν for polymer concrete of compressive strength 58 MPa. During these tests they determined E to be about 18 GPa and ν to be about 0.22. It should be noted that fibre reinforcement was not employed in their specimens. The effect of fibre reinforcement is thought to result in a slight increase in E.

3. EXPERIMENTAL METHODS

It was realised from the beginning that the strength properties of polymer concrete are likely to be subject to significant statistical variation for the reasons pointed out in sections 2.1 and 2.3.2. For this reason at least ten specimens have been produced for each type of polymer concrete mix investigated. In certain cases 20 to 30 specimens of each type are considered necessary to ensure reproducibility of results, and to draw relevant conclusions.

3.1 Selection of testing procedure

Common strength testing configurations used for materials characterisation are tensile, compressive, flexural, and impact testing. Of these methods it seems clear that flexural testing procedures are the most appropriate for polymer concrete. The reasons for this are as follows: -

- Polymer concrete is extremely abrasive, and is thus difficult to machine. A significant amount of machining is required to manufacture tensile test specimens, and it was thus accepted that it would be too time consuming to produce tensile test specimens in the required volumes. Gripping of polymer concrete in a tensile testing rig also presents problems.
- Flexural testing of specimens simulates the type of loading experienced by manhole covers in service.
- Compressive test specimens, although easier to manufacture than tensile test specimens, still present problems. In addition, the loading condition does not simulate the type of loading experienced by manhole covers in service.
- Impact loading was considered, but for preliminary characterisation of the material it was concluded that static testing would be likely to yield more useful results than impact testing methods.

Flexural testing methods have been well researched, and are well understood. There are two main modes of flexural failure. These are shear (in short thick specimens), and bending (for longer, thinner specimens). When the specimen is ‘thick’, shearing forces are the dominant action, since the thickness restricts the deflection of the specimen. In thinner specimens, bending action can occur, and the specimen can deflect since it is not as restricted by its thickness. For this reason, specimen geometry must be carefully selected to ensure that failure occurs in the desired mode.

Common flexural testing can be performed in two main configurations. These are centre point loading, and four point loading. The configurations are illustrated.

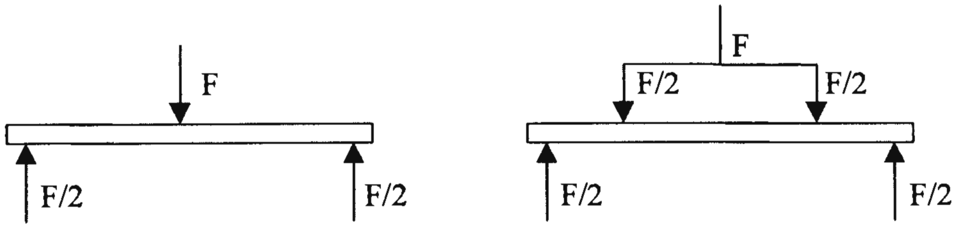
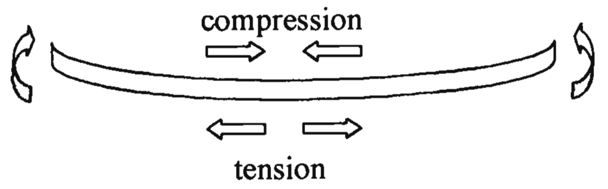


Figure 3.1.1 Illustration of centre point and four point flexural loading.

The configuration of flexural testing results in the top surface of the specimen being subjected to compressive forces, while the bottom surface experiences tensile forces. The stress distribution is separated by a neutral axis parallel to both surfaces.



Note : Curved arrows indicate the direction of the applied moment

Figure 3.1.2 Illustration of compressive and tensile forces in flexure.

In centre point flexure the forces are more concentrated at the centre of the beam than in four point bending, where the applied force is more spread out. From basic beam theory it is possible to calculate the stress distribution in a flexural test specimen, knowing the applied load, and the geometry of the specimen.

Beam theory predicts (for the case of pure bending): - $\sigma = \frac{My}{I}$

where M is the applied moment, and $M_{\max} = \frac{FL}{4}$ (L is the span of the test)

y is the distance from the neutral axis

I is the second moment of area of the beam section

The value of stress calculated in this way from a three point bend test is often referred to as the ***modulus of rupture*** (MOR) of the material.

3.2 Selection of specimens

Beam-shaped specimens were selected for the flexural testing configuration. The dimensions for these beam specimens were decided upon after the consideration of the following factors: -

- Specimens should be large enough to be representative of a polymer concrete mix.
- Moulding is easier with larger specimens.
- Specimens should be subjected to bending rather than shear forces. This factor determines the relevant span/thickness ratio, which is usually between 8 and 10.
- Experiments were to be carried out using large aggregates. Theory predicts that to be representative, the size of the largest aggregate in a sample should be no more than one third the size of the smallest dimension of the sample.
- The capacity and size of the testing machinery and rigs available were also considered.

After consideration of these factors, the following specimen shape was decided upon: -

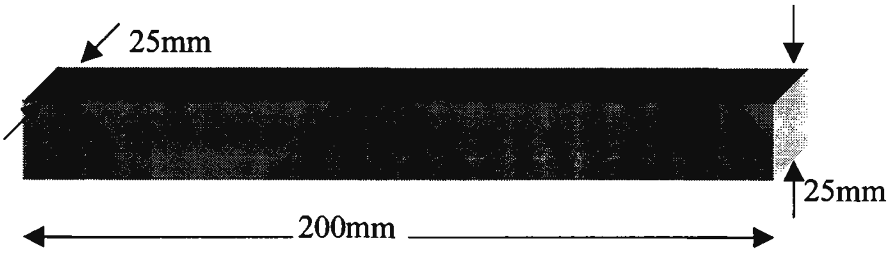


Figure 3.2.1. Chosen specimen shape and dimensions.

A three point flexure configuration was decided upon, mainly because a testing rig large enough to accommodate these specimens in four point bending was not available. The testing machine to be used was a Zwick Universal Testing Machine (UTM), and the loads were to be applied by means of short cylindrical steel bars mounted in V-shaped locators. Resilient leather shims were to be used in some cases, to avoid high surface stresses on the loading points. The configuration is illustrated in figure 3.2.2.

It is suspected that a small amount of shear force may be present during this testing procedure, since the span/thickness ratio is about 6 (the span is 140 mm). This is then not a case of *pure* bending, however the possible specimen sizes were influenced by other factors as well, and the problem turned out to be unavoidable. It has been found that the size of these shear forces, although not zero, are small and the bending equation gives a reasonable approximation. Any error here does not affect the *comparison* of specimen strengths, since constant conditions are employed.

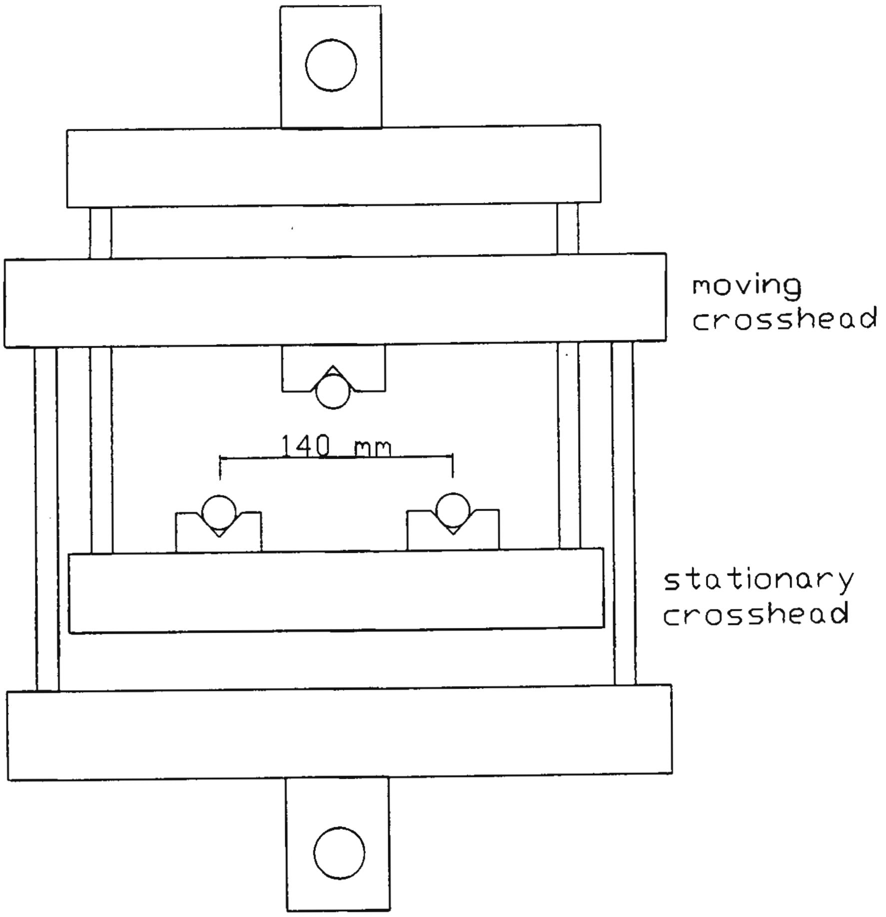


Figure 3.2.2. The rig employed for flexural testing of specimens.

The above rig was then mounted in the UTM. It is clear that tension on the ends of this rig, induces compression in the inner parts of the rig. The three V – points induce three point flexure on beam shaped specimens. Due to the available configurations, the distance between the centres of the V – points was chosen to be 140 mm. The load-deflection curve can be analysed by the use of standard spreadsheet packages, and failure loads can thus be recorded. A mould for the manufacture of ten specimens at a time became available during the course of the research, and greatly simplified matters. The next few sections deal with the equipment, and method to be used, for various experiments that were to be conducted.

3.3 Preliminary tests

Flexural testing of specimens was carried out in an attempt to characterise the material. Some variation of mix proportions and additions was employed in order to understand the benefits of certain components of polymer concrete mix.

1) Tests on standard polymer concrete were performed in order to determine the basic strength of the material. Flexural test specimens were made up and tested at a constant deflection rate of 3 mm/min until final fracture.
2) Flexural tests were carried out to determine the strain rate sensitivity of the material. Deflection rates were varied from 1 mm/min up to 60 mm/min, and the initial and final fracture loads were recorded.
3) The effect of chopped strand fibre additions to polymer concrete was investigated. Specimens containing no chopped strands were made up and tested in flexure at a common deflection rate of 3 mm/min. The results were then compared to those of specimens containing standard amounts of chopped strand fibres.
4) The effect of woven roving reinforcement was considered. Specimens were produced that incorporated a layer of woven roving mat at a position approximately one-third of the thickness from the bottom surface (tension side). Specimens were tested in flexure at a common strain rate of 3 mm/min, and the results compared to those of specimens without reinforcement.
5) Fracture toughness experiments were also carried out. Standard specimens were produced, and a 5 mm notch was machined into the bottom surface (tension side). Specimens were then placed in flexure, and the load-deflection curve was analysed. The fracture toughness was then calculated using the following modified form of the Griffith equation[24, 38]: -

$$K_{IC} = \frac{6M_c a^{1/2}}{bw^2} Y \qquad (3.1)$$

where M_c is the applied critical bending moment

a is the notch depth

b is the width of the specimen

w is the thickness of the specimen

$Y = 1.93 - 3.07(a/w) + 14.53(a/w)^2 - 25.11(a/w)^3 + 25.80(a/w)^4$

6) Tests were carried out to determine if the resin content could be reduced by the removal of chopped strand fibres from the mix (see point 3). For this experiment specimens were manufactured without chopped strand fibres, but also containing only 15 % resin by weight instead of the usual 21-22 %. These specimens were then tested in flexure at a common deflection rate of 3 mm/min.

All of the above tests are considered to be preliminary ***characterisation*** tests. Each test involved 10 to 20 specimens. All further flexural tests were performed at a common deflection rate of 3 mm/min. The reason for adopting this method was that the strain rate sensitivity had already been established, and constant conditions were required for any further tests. It was realised that there was a risk of introducing too many variables into the testing procedure, which could hamper further investigation.

In addition to the above tests conducted, the performance of the rig was assessed at this stage. Although the results obtained from the specimen shape used were considered to be reasonable, a slight modification to the specimen shape was decided upon in order to reduce scatter in individual results. The modification involved introducing a small chamfer onto the edges of the bottom surface with an abrasive sanding machine. This adjustment was found to be necessary since there were significant voids and pores in these areas that could increase the stress concentration of the edge itself.

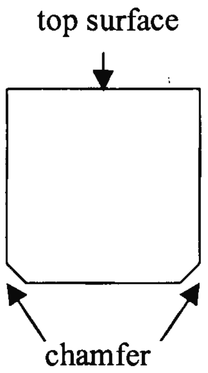


Figure 3.3.1 Cross section showing the chamfer introduced to the specimen shape.

3.4 Assessment of different resins

Part of the research involved an assessment of different resins available on the market. The standard resin used almost exclusively by AV Mouldings for the manufacture of polymer concrete manhole covers is the orthophthalic polyester, UPE1623. Its properties and performance in polymer concrete are well known. However, interruptions in supply often result in decreased production (with associated financial implications). For this reason, it is desirable to make use of more than one resin supplier, ie. one or another could be called upon in the event of supply interruptions. Before attempting to manufacture polymer concrete using any different resins, the following properties of the resins needed to be considered: -

1) Price
2) **Performance in polymer concrete**
3) Viscosity (affects workability of mix)
4) Ease of use, (pre-accelerated etc.)
5) Storage life
6) Gel time
7) Safety, health hazards, irritation of skin, styrene emission etc.

During the course of the research conducted, the following resins were considered to be potential alternatives to the standard resin used: -

- Enydyne dicyclopentadiene resin
- Polylite 2917 (orthophthalic)
- Polylite 33-410 (isophthalic)
- Polylite 2535 (orthophthalic)
- Crystic 197 (orthophthalic)
- Crystic 272 (isophthalic)
- Crystic waxed (orthophthalic)

The factors mentioned above were all considered when assessing these resins, but by far the most analysis was given to point number 2. Test specimens were made up from mixes using the above resins in standard proportions, ie. the only variable was the type of resin used for the mix. During the course of the work, the characteristics of the wet mix, and the hardened specimens were noted. Useful information was also obtained from staff on site at the AV Mouldings factory regarding the desirability of the different resins. Their main concerns are the skin irritation, and styrene emission factors. Other points such as workability of the mix and gel time were noted.

3.5 Degradation experiments

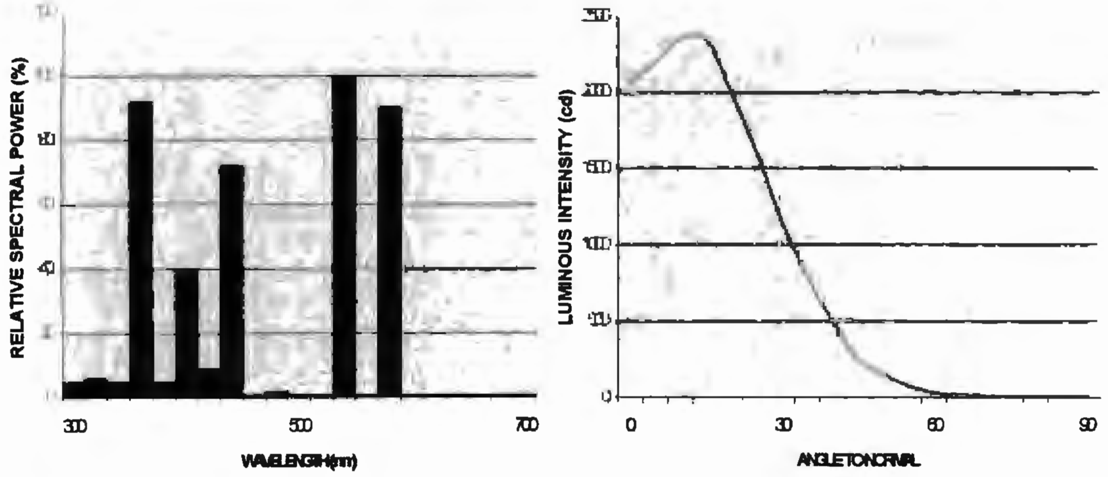
Investigation of the degradation effects of UV radiation, and aggressive chemical environments was considered to be an important aspect of the research.

3.5.1 UV Degradation

In the case of UV radiation experiments, specimens were exposed to UV radiation for varying periods, in an attempt to determine how time of exposure affects the strength properties of polymer concrete. The specimens used for these experiments were made from standard mix (using UPE1623 orthophthalic polyester resin). UV radiation was produced by a 125 W mercury vapour lamp, emitting radiation of nominal wavelength 300 – 400 nm (see figure 3.5.1). This range of wavelengths is considered suitable, since shortwave UV radiation (< 300 nm) although produced by the sun, is mostly absorbed by the ozone layer in the Earth's atmosphere. The objective was to simulate the effect of the sun on polymer concrete (in an accelerated test).

The lamp was mounted in a closed structure with reflective foil paper on the inside walls. A cooling fan was also mounted in the structure to ensure ambient temperatures prevailed inside. The distance from the lamp to the floor of the structure was calculated to ensure that the maximum power angle from the lamp would cover all polymer concrete specimens inside. The positioning of the specimens was also selected such that all specimens received (more or less) the same amount of radiation.

For the preliminary tests, 24 standard specimens were placed in the UV 'cupboard', and irradiated. After two weeks constant exposure, six specimens were removed. After four weeks, the next six specimens were removed etc. When the test was completed, four groups of six specimens had been obtained, with exposure times of two, four, six, and eight weeks. All specimens were then tested in flexure on the same day, to ensure that they were all of the same age. These results are then compared to those of control specimens produced from the same mix as the test specimens.



(a) *(b)*

Figure 3.5.1 (a) Plot of radiation wavelengths from lamp, (b) plot of power vs angle from centre of lamp

The results obtained from this test were very interesting, and led to further tests being conducted. Essentially the second test procedure was the same as the first, but the intervals of time used were two, five, seven, and nine weeks for each group of specimens respectively. Ideally, it would have been desirable to have had groups of ten or more specimens each, rather than six, but unfortunately this number was determined by the size of the UV 'cupboard', which in turn was determined by the nature of the UV lamps available. However, six specimens are considered to be sufficient for the purposes of this experiment. In addition, the maximum times of exposure were determined by the useful working life of the UV lamp. The rating was that after 2000 hours the power output would be at 80 %. For this reason, it was decided that 2000 hours operation time would have to be the limit, to ensure relatively constant conditions.

It is important to assess the reproducibility of the results obtained in the preliminary UV radiation test. This is the main reason why the second UV experiment was conducted. The results are presented in section 4.3.

3.5.2 Degradation by solvents

The following solvents are considered relevant to the investigation: -

- Water – eg. rainwater, groundwater, spillages
- Gasoline – eg. spillages, leaks
- Diesel Fuel – eg. spillages, leaks
- Engine Oil – eg. spillages, leaks
- Saline solution (solution rather than a solvent) – eg. seawater, salty atmosphere

The above solvents were decided upon, since they can conceivably come into contact with polymer concrete manhole covers. Other more complex solvents are not considered relevant to the investigation. The preliminary testing procedure involved the immersion of groups of specimens in each solvent for varying times.

The time intervals selected were one, two, and three weeks. All specimens were weighed prior to immersion in the solvents. After the relevant times, six specimens were removed from the solvent baths, and immediately weighed. After drying they were re-weighed. Specimens were then stored until completion of the experiments, along with control specimens (from the same original mix) that had been kept aside.

Specimens were examined for any visible signs of deterioration, and then tested in flexure all on the same day, under constant conditions. The results of these tests were then compared to those of the control specimens.

Of particular interest is the effect of water and saline solution on the mechanical properties of polymer concrete. For this reason two long exposure experiments were set up, one involving water, and the other saline solution. The time of exposure was approximately seven months.

All results from these experiments are presented in section 4.4.

3.5.3 Degradation by acids and alkalis

The procedure for investigation of the effect of acids and alkalis was similar to the procedure used for degradation by solvents. The substances used in these tests were the following: -

- 10 % H_2SO_4 solution (acidic, pH 1.5)
- 10 % HCl solution (acidic, pH 1.5)
- 10 % NaOH solution (alkaline, pH 13)

These solutions are considered to be highly concentrated compared to the concentrations that could be expected from acid rain and alkaline ground water etc. but for the purposes of an accelerated test they were acceptable. The main reason for this decision is that the tests were intended to determine the *extent* of deterioration possible.

Groups of six specimens were exposed to the solutions for periods of five, ten, and fifteen days respectively. All were weighed prior to immersion. After the required times of immersion, specimens were removed from the solutions and immediately weighed. After drying they were re-weighed, examined for visible signs of deterioration, and then stored in ambient temperatures until all exposures had been completed.

Specimens were then tested in flexure, and the results compared to those of control specimens from the same mix batch, under constant conditions.

A second set of experiments were conducted in the same manner as before, in order to determine the reproducibility of the results. The exposure times used are slightly shorter than in the first test.

The results from these experiments are presented in section 4.5

3.6 Postcuring experiments

The process of postcuring a hardened resin is discussed in section 2.3.1. The purpose of postcuring is to ensure that the resin is completely crosslinked, or polymerised. It is usually carried out by exposing hardened products to elevated temperatures for short times (hours). In most cases resin manufacturers recommend that postcuring be carried out on finished products, since optimum properties of the products can be achieved through this process.

In order to investigate the effect of postcuring procedures, a batch of ten specimens were produced from the same mix (ensuring exactly equal amounts of catalyst had been added to all hardened specimens). Specimens were allowed to cure at ambient temperatures for at least 24 hours. Five specimens were kept at ambient temperatures, and five were postcured in an oven at 80° C for three hours, according to the resin manufacturer's recommendations. The lay-man's way of determining the state of cure of a resin system, is often by the residual smell of the product. Indeed it was noticed that the postcured specimens had virtually no residual smell, while the non-postcured specimens did have a noticeable smell.

All specimens were then tested in flexure, and the results of the postcured specimens were compared to the results from the non-postcured specimens. Particular attention was paid to the *type* of failure of the specimens. Postcuring is reported to increase the strength as well as the *rigidity* of hardened resins, possibly introducing a certain amount of brittleness to the material (in flexural mode).

An additional three sets of experiments involving postcuring were carried out after the results from the above test were analysed. The reason for this was to assess the validity and reproducibility of the results. All experiments involved postcuring at 80° C for three hours. Variation of this procedure was not considered, since it is well known that higher temperatures require shorter times, and lower temperatures require longer times, ie variation of times and temperatures do not affect the *state* of cure of the resin (if performed correctly). The results from all of these experiments are presented in section 4.8.

3.7 Effect of silane additions

As discussed in section 2.3.3, it is well known that silane additions to polymer concrete mix generally improve the mechanical strength of products produced from these mixes. The evidence for this is rather sketchy, however, and thus a full investigation was required. After consultation with various chemical companies regarding the grade and type of silane to use, a sample was acquired from Chemimpo (Pty) Ltd. The product code was Silane GF31.

In order to investigate the effect of silane additions, a few preliminary experiments were carried out. In the first, 1 % silane (by weight of resin) was added directly to the resin before aggregates and fibres etc. In the second experiment, 1 % silane was added to the *aggregate*, and thoroughly distributed through it. The aggregates were added to the resin and the mix procedure was then conducted in the standard way. Since published research on the subject refers to silane treated resin, as well as silane treated aggregates, it was considered relevant to investigate both procedures.

A more thorough investigation was then performed. Five standard mixes were made up containing 0.25, 0.5, 1, 2, and 5 % silane. Due to the fact that silane products are generally expensive, it seemed relevant to determine how improvements in mechanical strength of polymer concrete vary with the amount of silane added. In this way the minimum silane addition for a useful improvement in strength can be determined. Fracture toughness experiments were also carried out on notched beam specimens incorporating silane in order to determine if silane has any beneficial effect on K_{IC}. During the course of the work, all relevant mix characteristics such as workability and gel time etc. were noted.

All specimens were tested in flexure at 3 mm/min. The mode of failure was of particular interest, since it has been reported that silane additions add rigidity as well as strength to polymer concrete.

Results from these tests are presented in section 4.7.

3.8 Aggregate grading experiments

It is well known that aggregates graded to give a minimum void content improve the mechanical properties of polymer concrete products. As discussed in section 2.2.2, it is also possible to obtain polymer concrete mixes with lower resin contents, effectively reducing the cost of the material. Substantial research has been conducted in order to understand how this can be achieved. However, it is questionable whether previous work can be applied to the polymer concrete material concerned in this investigation, mainly since previous research involved materials without chopped strand fibre reinforcement. For this reason, the effect of aggregate grading was investigated in considerable detail.

After investigation into the types of aggregates available, the following different aggregates were obtained from Consol Industrial Minerals (Pty) Ltd.: -

Grade	Particle Size (mm)	Price (R/ton)
No. 2 HP Sand	0.075 – 0.212	254
No. 1 Grade	0.1 - 1	122
7/16 Filter Grit	1.2 – 2.4	790
4/10 Filter Grit	2.4 – 4.8	790
5-9 mm Filter Grit	5 - 9	790

Table 3.8.1 Different aggregates used in polymer concrete mixes. All grades are listed under their product names. Prices valid July 1998 – June 1999.

For the purposes of this investigation, No.1 Grade and No.2 HP Sand are denoted as fine aggregate. The Filter Grits are denoted as coarse aggregates. No.1 Grade is the aggregate that AV Mouldings use for their polymer concrete products. The sand is supplied in a washed and dried form, and is relatively free of contaminants. It is also the cheapest quality aggregate available, since it can be supplied in bulk. Other aggregates are supplied in 25 kg and 40 kg bags. The Filter Grits listed in table 3.8.1 are substantially more expensive since they are not locally produced.

An important point that was noticed was that the 5 – 9 mm Filter Grit contains particles that are too large for the specimen size being used. Theory suggests that to be representative, a sample should not contain particles of sizes larger than one third of it's smallest dimension. For this reason a 9 mm particle is too large to incorporate into a 25 X 25 X 200 mm bar specimen. All particles larger than 6.7 mm were thus removed from the aggregate. The choice of 6.7 mm as the limit was determined by a small safety factor that was incorporated, as well as the sizes of sieves available.

For the purposes of this investigation, the following aggregate blends were decided upon: -

Mix	No. 2 HP Sand	No.1 Grade	4/10	7/16	5 - 6.7 mm
1	11.5	14		74.5	
2	14	74.5		11.5	
3	33	33		33	
4	25	25		50	
5	15	20	20	30	15
6	15	40	15	15	15
7	50	50			
8	10	45		45	

Table 3.8.2. List of aggregate blends selected for testing. All values refer to weight percent of total aggregate.

The aggregate blends listed above were chosen initially to determine the influence of aggregate containing mainly coarse or fine particles. Mixes 3 and 4 give a roughly even spread of particle sizes. Mix 5 consists of mainly coarse aggregates, but not only of one size as in Mix 1. Mix 7 gives a fine aggregate distribution. Essentially, the No. 2 HP Sand in this mix acts as a microfiller, although traditionally microfillers consist of even smaller particles.

The procedure for experimenting with these aggregates was kept constant. For each particular mix, the aggregates were first prepared by weighing them and mixing them in the relevant proportions (all aggregates were supplied in a washed and dried condition, thus no further drying was considered necessary). The aggregates were then mixed into the resin/fibre matrix. The percentage resin used was the standard amount of 21 – 22 %, and all other additions were also added in standard proportions.

The nature of the wet mixes was carefully noted during the work. Groups of ten specimens were moulded for each particular mix and allowed to set. Flexural testing of these specimens commenced after suitable curing times (in most cases at least 7 days were allowed).

A second set of experiments commenced in order to verify the results obtained. The procedure followed is the same as for the first set of experiments.

After completion of all flexural tests, samples were taken from the specimens, and examined using stereo optical microscopy.

All results are presented in section 4.6.

3.9 Vibratory moulding techniques

The purpose of vibrating the wet mix in moulds is an attempt to allow air bubbles that may have been incorporated into the resin during mixing to escape. At the same time it allows the mix to settle properly into the moulds, improving the density of the final product. Experimental procedures were planned in order to investigate this effect.

A vibrating table was acquired upon which the mould for specimen production could be placed. The vibration frequency of the table was set at 56 Hz, and unfortunately this variable could not be changed. The mould was tightly fastened to the table using G-clamps, and standard polymer concrete mixes were then prepared.

In the first experiment, catalysed mix was placed in the moulds and compacted by hand. The vibrating table was then switched on for approximately five minutes. This time of vibration was decided upon, since once the mix had begun to set the effect of vibration would not further improve the properties of the final hardened material. After five minutes the vibrating table was switched off, and the specimens were allowed to set fully.

In the second experiment, the procedure was the same as for the first, except that the vibrating table was switched on *before* the placement of the mix in the moulds. The vibration time then was approximately 10 minutes. Also, a mix was made up containing 1 % silane addition. This mix was then moulded according to the previous procedure.

Specimens from all sets of experiments were examined after removal from moulds, in order to determine if any improvements were visually detectable. Specimens were then tested in flexure under constant conditions.

After fracture, certain specimens were selected for more detailed analysis. Cross sectional samples were cut from these specimens, and analysed using stereo optical microscopy. Conformances as well as irregularities were noted.

Results and from these vibratory moulding techniques is presented in section 4.9.

3.10 Design alterations

Research and development was requested by AV Mouldings regarding the refinement of the design of their Type 2A replacement cover. At the time the Type 2A design was based on the design for cast iron covers. However, the properties of polymer concrete vary vastly from those of cast iron, and it is questionable whether this design can be used successfully for polymer concrete covers. This does not necessarily imply that polymer concrete is unsuitable as a material for manhole covers, but it does mean that redesign of the Type 2A replacement cover may be required. Indeed, the majority of product failures involve Type 2A replacement covers designed on the basis of cast iron cover specifications.

Redesign of the Type 2A replacement cover is discussed in section 6, however experimental methods are needed to assess the redesign. As mentioned in section 2.3, the calculation of stress distributions in polymer concrete manhole covers becomes quite complicated. For this reason actual covers are to be cast incorporating design changes that were arrived at during the redesign phase of this research. All mixes used in these experiments should be standard, and the moulding procedure should be performed as per usual.

Testing of these covers is to be conducted according to procedures described by SABS 558:1973 first. Products incorporating design alterations are then to be tested in service as well, in order to assess their performance.

4. RESULTS

The results obtained from the various experiments conducted are presented in this section. Results are generally structured in the same order as they were set out in section 3. Basic explanations are given here, but the method used to assess the validity of the results and full discussion is presented in section 5.

Due to the significant variance in strength values obtained in many of the tests performed, results from individual specimens as well as average values and standard deviations are tabulated in some cases. Initially only the failure loads are compared, since these give a good indication of the strength of the material. Graphs showing the observed trends are included as well.

4.1 Preliminary tests

The results of the preliminary tests described in section 3.3 are presented here. In essence, they convey general information about the mechanical behaviour of polyester polymer concrete using one type of aggregate, and the effects that its components produce in the final product.

4.1.1 Basic strength of specimens

These tests were performed on specimens of *standard polymer concrete mix*. The mix proportions that were used are described in section 2.5.1. A typical flexural load-extension curve from these tests is shown in figure 4.1.1.

It is found from these tests, as well as from control specimens for further experiments that the average failure load for standard polymer concrete mix is about 1850 N. The standard deviation in this value is 110 N. It should be noted that this deviation value is certainly not insignificant. The strength results from individual test specimens of a batch of 20 specimens are summarised in table 4.1.1. All further flexural tests on standard polymer concrete mix confirm that a strength value of 1850 N can be taken as the basic average flexural strength for specimens in this configuration.

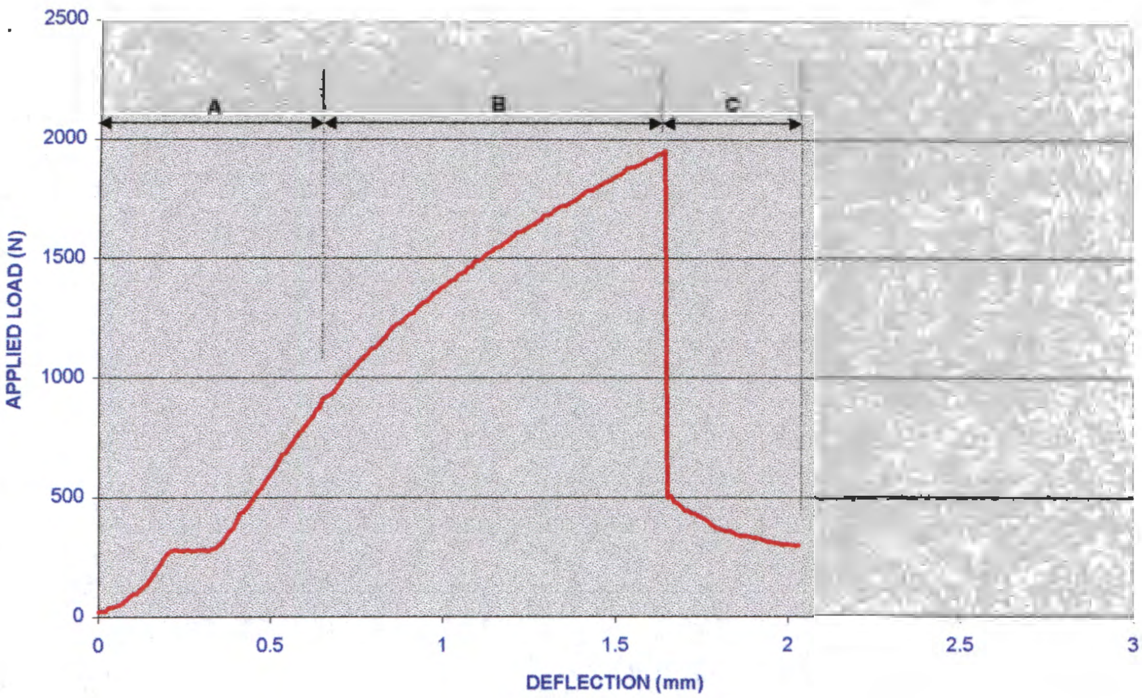


Figure 4.1.1. Typical flexural test curve for standard polymer concrete specimens.

The flexural test curve for polymer concrete comprises three main regions. These are the elastic region, A, a region of non-linear deformation, B (permanent set), and the region after failure during which a certain amount of residual strength is exhibited, C. The slope of the graph in region A gives an indication of the modulus of the material. The slight 'step' formation in region C is typical of the flexural test curves for polymer concrete. Examination of the above test curve reveals that there appears to be a slight levelling off effect in region A, at a load value of approximately 300 N. The effect is observed for the vast majority of all flexural tests performed. Although it does not seem to affect the maximum failure load of the specimen, it is nevertheless considered to be a phenomenon that should be explained. The conclusion is that the levelling off effect can be attributed to the load carrying points on the rig settling into the specimen slightly during testing. This conclusion was verified by examination of the specimens after testing. Small indents were visible at the load points. The most important values on the graphs obtained are the maximum failure loads, and as a result not much further attention is given to the 'levelling off' effect. The toe end of the curve exhibits the shape of a typical flexural test curve. The effect is due to self alignment of the specimen during the start of the test.

No.	Failure load (N)	No.	Failure load (N)
1	1820	11	1930
2	1910	12	1860
3	1650	13	2010
4	1930	14	1790
5	1900	15	1880
6	1680	16	1940
7	2050	17	1690
8	1740	18	1820
9	1880	19	1750
10	1770	20	1910
		Ave.	1845
		STD	110

Table 4.1.1. Typical individual test results from one batch of standard polymer concrete specimens

4.1.2 Strain rate sensitivity

A graph illustrating the effect of strain rate during flexural testing is presented in figure 4.1.2. Four different strain rates (1 mm, 3 mm, 15 mm, and 60 mm/min) were employed during this test. Strain rates are reflective of the rate of motion of the upper load point on the flexural testing rig. This translates approximately to the rate of central deflection of the specimen. Each point on the graph in figure 4.1.2 is an average value of ten specimens, and the standard deviations all fall in the region of 120 – 150 N. It should be observed immediately that there is not a significant difference in failure loads for specimens tested at 1 mm, 3 mm, and 15 mm/min. However, a significant increase is noted for specimens tested at 60 mm/min.

All further flexural testing was performed at 3 mm/min. This value is considered appropriate since it allows for equilibrium loading, and due to the large number of tests performed, is convenient to employ since each test lasts typically about 2-3 minutes.

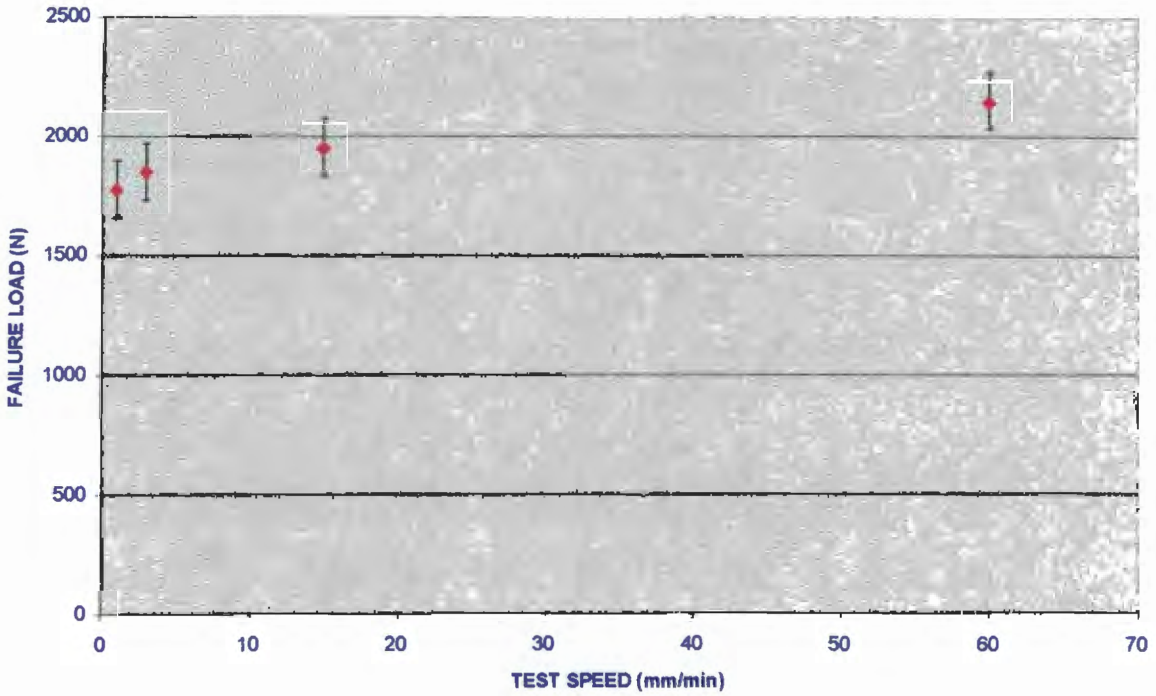


Figure 4.1.2 Strain rate sensitivity of standard polymer concrete specimens.

4.1.3 Effect of chopped strand fibre additions

These tests involved the testing in flexure of specimens made from standard mixes, one containing 0.5 % chopped fibres (12.5 mm and 18 mm in 50/50 proportion), and the other containing no chopped fibres at all. Tests reveal that specimens containing chopped fibres are significantly stronger in flexure than specimens containing no fibres.

The effect of the volume fraction of chopped strand fibres added to the mix was not investigated. It is known that higher amounts of fibres in the mix increase the viscosity of the mix, and thus a higher resin content is required to achieve a workability similar to that of a mix containing standard amounts of fibres. The issue of volume fraction of fibres is thus considered to be relatively well understood and becomes one variable too many to deal with.

The results from individual tests reflecting the effect on polymer concrete strength of fibre additions are presented in table 4.1.2.

No.	Fibre reinforced	Not fibre reinforced
1	1820	1600
2	1910	1500
3	1650	1520
4	1930	1550
5	1900	1570
6	1680	1560
7	2050	1540
8	1740	1590
9	1880	1550
10	1770	1510
Ave	1835	1550
STD	125	33

Table 4.1.2. Individual results of standard specimens showing the effect of the chopped fibre additions to polymer concrete. All strengths are given in Newtons.

The above table includes ten strength values from table 4.1.1 for comparison. The lower failure load for non-fibre reinforced polymer concrete is readily noticed. The average difference in failure load is approximately 25 %. In addition, it should be noted that the standard deviation of values for non-fibre reinforced polymer concrete is much lower than that for fibre reinforced materials. This upholds the theory discussed in section 2.3.2. "Fibre reinforcement", that the addition of fibres substantially increases the strength of polymer concrete, but also introduces a considerable variation to this (improved) strength.

The introduction of fibre reinforcement to a polymer concrete mix improves the toughness characteristics of products as well. Specimens made from fibre reinforced polymer concrete exhibit a certain amount of 'residual' strength after initial fracture, while specimens containing no fibres fracture completely at the failure load. Figure 4.1.3 shows how the fracture of reinforced specimens requires the pullout, or fracture of fibre reinforcement, which significantly increases the strength and toughness of the material.

It was also noticed that the mix containing no fibre reinforcement was 'wetter' than the mix that did contain fibre reinforcement. This observation was expected, and does affect the amount of resin required to achieve a workable polymer concrete mix.

Figure 4.1.3. Photograph of crack propagation through fractured specimens with chopped strand fibre reinforcement. Magnification 18X

4.1.4 Effect of woven roving reinforcement

Woven roving reinforcement significantly improves the mechanical strength of polymer concrete in flexure. Standard specimens were made up containing woven roving at a distance approximately one third of the thickness from the tension side of the specimens. Flexural strengths are improved by almost 30 % in some cases. Table 4.1.3 presents the results of flexural tests on these specimens compared to those of non-reinforced specimens.

In the same way as chopped fibre additions, woven roving reinforcement provides residual strength to the specimens after initial fracture. In this experiment the specimens without the woven roving reinforcement, do contain chopped fibre additions, and so do display residual strength. However, the woven roving reinforcement (in addition to the chopped fibres) further improves the residual strength and toughness of the material.

Examination of fractured specimens revealed that some of the woven roving fibres had not fractured. Rather, they had been 'pulled' out of the matrix. This seems to indicate that the strength derived from the woven roving reinforcement is not only dependent on the strength of the fibres themselves, but also on the strength of the bond between the fibres and the matrix. For this reason good impregnation of the woven roving matting by the resin is required to achieve these strength gains. Further detailed discussion of this topic is included in section 5.6.2.

No.	Reinforced	Not reinforced
1	2050	1820
2	1980	1910
3	1940	1650
4	2500	1930
5	2480	1900
6	2250	1680
7	2080	2050
8	2390	1740
9	2000	1880
10	2280	1770
Ave	2195	1835
STD	210	125

Table 4.1.3. Effect of woven roving reinforcement on flexural strength of polymer concrete specimens. All strengths are given in Newtons.

It should be noted that the strength gains are subject to a good deal of variation. This is because it is impossible to ensure that exactly the same pattern of fibres are incorporated into each specimen. Differences are inevitable, and the combined effect of the chopped fibres and the woven roving matting result in significant statistical variation. However, it is absolutely clear that these reinforcements give considerable improvements to the strength and toughness of polymer concrete.

4.1.5 Fracture toughness

The fracture toughness of standard polymer concrete with chopped strand reinforcement but no woven roving matting, is measured as the average of ten tests performed on notched beam specimens of standard size. The notch length used in all cases is 5 mm. Substituting the relevant values into equation 3.1, the fracture toughness (K_{IC}) is calculated to be 1.85 $MNm^{-3/2}$. The deviation in this value is approximately 0.27 $MNm^{-3/2}$. This calculation is based on an average failure load for notched specimens of 1120 N with a standard deviation of 165 N.

The fracture toughness of specimens containing no chopped fibre reinforcement is measured in the same way. The average failure load for these notched specimens is found to be 705 N with a standard deviation of 22 N. Equation 3.1 gives K_{IC} for these specimens as 1.17 $MNm^{-3/2}$, with a standard deviation of 0.03 $MNm^{-3/2}$.

These values can be compared with those of ordinary concrete and cast iron. The fracture toughness values of ordinary concrete made from Portland cement and cast iron are 0.2 – 0.5 $MNm^{-3/2}$ and 6 – 10 $MNm^{-3/2}$ respectively*. The fracture toughness of plain cured polyester resin falls in the range 0.5 – 1 $MNm^{-3/2}$.

The results serve to verify that the fracture toughness of polyester polymer concrete can be improved by the addition to the mix of chopped strand fibre reinforcement.

4.1.6 Effect of resin content

An attempt was made to reduce the resin content of polymer concrete by eliminating the chopped strand fibres. It has already been established that chopped strand fibre additions to polymer concrete decrease the workability of the wet mix significantly. By doing away with the chopped fibres it is possible to produce a polymer concrete mix with roughly the same workability as a mix containing chopped fibres, but using only 15 wt % resin as opposed to the standard 21 wt %. Table 4.1.4 presents the results of this experiment. It should be noticed immediately that the average failure load for the mix containing 15 % resin without chopped strands (Mix 2) is approximately 33 % less than that for the standard mix design (Mix 1).

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* Figures are quoted from Ref. 30 and 31. Different grades of cast iron exhibit varying fracture toughness, but for manhole covers low grade cast iron ($K_{IC} \approx 6$ $MNm^{-3/2}$) is usually used.

A significant part of this difference is likely to be due to the lack of chopped strand fibres in Mix 2. The reduction in resin content seems to be detrimental to the mechanical properties of *this type* of polymer concrete. It was observed in section 4.1.3 that a 21 wt % resin mix without chopped strand fibres results in a 20 % decrease in strength compared to that of the standard mix design.

No.	Standard	15 % resin (NC)
1	1820	1240
2	1910	1230
3	1650	1270
4	1930	1170
5	1900	1300
6	1680	1300
7	2050	1200
8	1740	1210
9	1880	1260
10	1770	1310
Ave	1835	1250
STD	125	45

Table 4.1.4. Comparison of strengths of standard mix to a 15 wt % mix containing no fibre reinforcement. All strengths are given in Newtons. 'NC' – no chopped strands.

It is also noticed that the standard deviation in strength of specimens made from Mix 2 is much lower than that for standard mix specimens. This difference is again attributed to the lack of chopped strand fibres in Mix 2, and conclusively proves that the addition of chopped strand fibres, while increasing the strength of polymer concrete, also introduces significant statistical variance to this strength.

These results indicate that it is not possible to reduce the resin content of this type of polymer concrete without affecting the workability of the wet mix. A possible solution that was envisaged at this stage is the method of using a graded aggregate system to reduce the void content of specimens made from polymer concrete mix.

4.2. Assessment of different resins

Six different resins were selected to be possible alternatives to the standard resin used (UPE 1623 orthophthalic). The properties of polymer concrete made from UPE 1623 were fairly well known at this stage, but the other resins had not been tested as a binder in polymer concrete. Various factors were considered when assessing these resins, but the most important factor is the performance of the resin in polymer concrete.

4.2.1 Strength performance in polymer concrete

The failure loads of groups of ten specimens made from polymer concrete incorporating each type of resin are investigated. Table 4.2.1 lists the various resins used along with their average failure loads and standard deviations.

Resin	Type	Ave. failure load (N)	STD
UPE1623	ortho	1850	120
Enydyne	DCPD	1750	135
Polylite 33-410	iso	1930	190
Polylite 2917	ortho	1650	200
Crystic 272	iso	1640	240
Crystic waxed	ortho	1700	180
Crystic 197	ortho	1850	250

Table 4.2.1. Flexural strengths of polymer concrete made from different resins. Mix proportions were identical.

Rather large standard deviations are noticed among the results from mixes made with some of these resins. However, not too much should be read into this since it has already been established that the main factor affecting the variation in results is the addition of chopped fibres. Furthermore, although variations in average strength are observed for specimens made from the different mixes, it is doubtful whether these are due to inherent properties of the resins themselves.

4.2.2. Toughness

All specimens appear to fracture by the same process. This process involves crack initiation at the centre span of the specimen, followed by fast crack propagation at least halfway through the thickness – in some cases the crack propagates three quarters of the way through the thickness of the specimen. In this condition the specimen can be considered to have fractured completely, however the specimens do exhibit a certain amount of residual strength, provided mainly by the fibre reinforcement present in the material. The last stages of a flexural test involve tearing and slow crack growth, with fibre pullout and fracture in some cases.

The size of the initial crack, together with the amount of tearing and fibre pullout after initial fracture is considered to be a good visual method of assessing the toughness of the specimens. Specimens made from Enydyne (DCPD) mix appear to exhibit a more brittle behaviour; in some cases the specimens fracture completely through their thickness by crack propagation, with no tearing or fibre pullout. The two isophthalic resins investigated produce differing results. Crystic 272 exhibits a lower flexural strength than Polylite 33-410. Also, specimens made up from mixes using these resins seem to display brittle characteristics.

4.2.3 Workability of wet mixes

The workability of the mixes made with the resins listed in table 4.2.1 was physically assessed. All the resins used have roughly the same viscosity at room temperature, and all mixes were made up using identical proportions of resin, aggregate, fibres and pigment. No significant difference in the workability of the wet mixes was observed. A method used to assess the viscosity of the mix, was observation of the amount of resin bleeding during moulding and curing. Again, no noticeable difference was observed for the mixes used.

4.2.4 General parameters

Consideration was given to parameters such as gel time, ease of use, cost, and health issues. This section presents the results of observations related directly to the resins themselves.

The price of isophthalic resins is generally higher than that of orthophthalic resins, thus it is of interest to determine how much better the isophthalic resins perform in polymer concrete at this increased cost. In general (see table 4.2.1), isophthalic resins do not exhibit improvements in mechanical properties that are worth the extra cost of employing them.

The issue of gel time (which is an important factor since it influences mould turnaround time and thus productivity), is not a critical property of a resin. The reason for this is that resins supplied by Scott Bader and NCS Resins were supplied in non-accelerated form. The correct amount of accelerator could be added to achieve any required gel time. Of course the addition of this accelerator does introduce an extra phase into the manufacturing process which lengthens the moulding cycle. However, it is reported that resins from these suppliers are available in pre-accelerated form, and can be re-engineered at short notice to meet customer requirements.

Health issues are considered to be one of the more important attributes of the different resins investigated. Most of the information regarding these properties was obtained from staff at the AV Mouldings factory. It was reported that resins from NCS Resins and Scott Bader seem to be more irritating to the skin than other resins investigated. The reasons for this are not entirely clear, since the basic molecular structures of the polyesters are roughly the same. It is thought that the styrene content of the resins can contribute to the skin irritation factor. It is also known that the organic peroxide catalysts used in conjunction with polyester resins are highly irritating to the skin. In fact all manufacturers of these products recommend that skin contact with organic peroxides should always be avoided.

A summary of these factors along with a final assessment of these resins is presented in section 5.7.

4.3 Degradation by UV radiation

This section presents the results of the extensive UV irradiation programme that was carried out on polyester polymer concrete specimens. The experimental methods involved are detailed in section 3.5. The results from these experiments are considered to be rather important in determining the durability of polymer concrete under prolonged exposure to the sun.

4.3.1 Test 1

The results of the first test carried out on polymer concrete specimens (of standard mix design) are presented in the figure below.

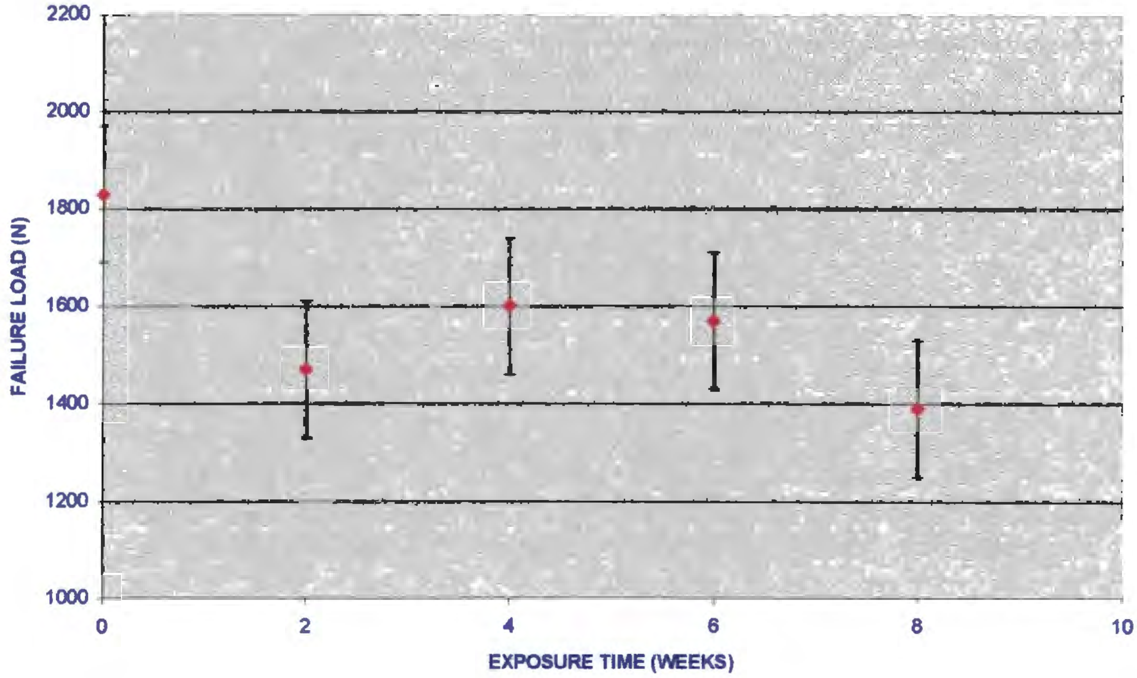


Figure 4.3.1. Graph of failure load vs UV exposure time for test 1.

The points on the above graph are averages of six results for each exposure time. The standard deviation of individual results is roughly the same as for non exposed specimens (125 N). It can immediately be observed that after eight weeks exposure, there is a significant drop in the average failure load. The drop after only two weeks exposure is also interesting. It is possible that this drop may be due to the experimental specimen configuration (such that these specimens received more radiation than others during their shorter exposure time).

Careful consideration was given to the validity of these results. In particular, the standard deviations in some cases overlap the range of individual results for other time exposures. For this reason, the exact positioning of the points on the graph are somewhat uncertain, and the result from one particular arbitrary test could lie anywhere within the ranges shown. However it is absolutely clear that degradation after eight weeks exposure produces a significant drop in flexural strength that cannot be attributed to scatter in individual results.

Specimens were examined for any visible signs of deterioration prior to testing. Close inspection revealed that the specimens (initially black) appeared to have lightened in colour somewhat. A faint yellowing was also noticed, in particular for specimens at longer exposure times. None of the specimens show any change in weight after the UV exposure periods.

Exposure time (weeks)	Average failure load (N)	STD (N)	% strength loss (N)
0	1830	130	N/A
2	1470	170	20
4	1600	180	13
6	1570	180	14
8	1390	120	24

Table 4.3.1 Analysis of results of UV radiation test 1.

The mode of failure of all specimens appears to be the same, ie. the sudden appearance of large crack propagating at least halfway through the specimen, followed by slow tearing and fibre pullout. Although the mechanism of UV degradation is thought to be embrittlement due to photo-oxidation of the polymer chains, it was not possible to *discern* any unusual brittle behaviour for any of the specimens.

4.3.2 Test 2

The results of the second UV exposure experiment are presented in the figure below.

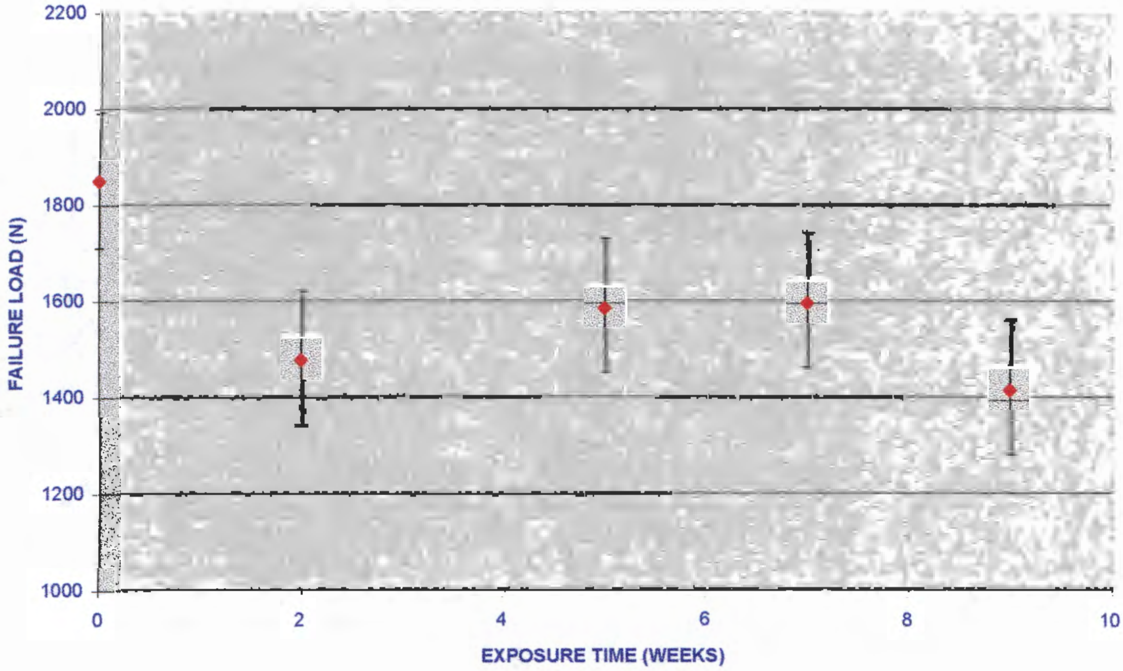


Figure 4.3.2. Graph of failure load vs exposure time for test 2.

As for test 1, the points on the above graph are the average failure loads for six specimens at each exposure time. For this test, exposure times of two, five, seven and nine weeks were employed. Comparison of figures 4.3.2 and 4.3.1 reveal that the same trends are evident in both cases. There is an initial drop in flexural strength after two weeks exposure, followed by an increase, and then another significant drop off. The standard deviations observed are of the order of 130 N (similar to unexposed specimens).

Again consideration has to be given to any overlap in flexural strength ranges for individual specimens. For this test there is a significant overlap, but there is no doubt that degradation did occur. The use of different exposure times to test 1 in this experiment was an attempt to 'fill in' the gaps in the graph for test 1. It is realised that slight differences in the mixes used for the two experiments could make a combination of the results for Test 1 and Test 2 invalid. However for the purposes of this experiment, any differences are considered to be negligible, since they are less than the inherent variation in strength of the individual results for each specimen. Results for test 2 are summarised in table 4.3.2.

As with test 1, all specimens were examined for any visible signs of deterioration. A faint 'yellowing' was observed, with an associated loss of colour. Again, none of the specimens show any change in weight after exposure to UV radiation.

Exposure time (weeks)	Average failure load (N)	STD (N	% strength loss (N)
0	1850	130	N/A
2	1480	180	20
5	1590	200	14
7	1600	170	13
9	1420	130	23

Table 4.3.2. Analysis of results of UV radiation test 2.

The same general trend is observed for test 1 and test 2, with the most severe loss of strength being recorded after UV exposure times of eight and nine weeks respectively.

4.4 Degradation by solvents

This section presents the results of degradation tests on polymer concrete using various different solvents. The results serve to determine the suitability for use of polyester polymer concrete products in certain environments.

4.4.1 Water

Specimens immersed in water for periods of one, two, and three weeks were weighed prior to, and after immersion. It was found that there was a small increase in mass directly after removal (due to water uptake) of approximately 0.5 %. However after drying, the specimens returned to their original masses. No sign of deterioration could be detected by direct visual inspection. Figure 4.4.1 summarises the results obtained from water exposure tests. The control specimen value is plotted at an exposure time of 0 weeks.

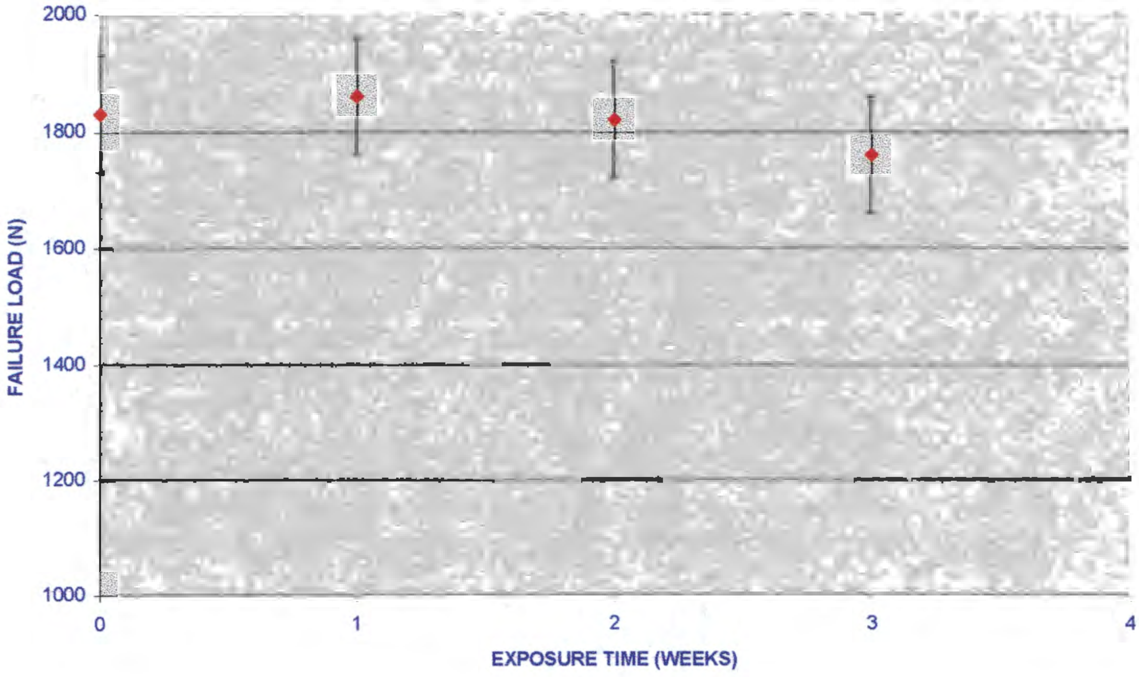


Figure 4.4.1. Graph of average failure loads vs time of exposure for water immersion tests.

It should be observed that there appears to be a slight drop off in strength after three weeks exposure to water. However, the range of values for each exposure time suggests that this decrease cannot be considered significant. In this case it is impossible to determine conclusively that there is any deterioration of the material for these exposure times at all. The mode of failure in all cases is similar to that of unexposed specimens.

A water immersion test was conducted for a period of seven months in order to determine if there was any deterioration in mechanical properties after long exposure times.

All specimens were removed from the water after the seven month testing period and immediately weighed. A small increase in mass was observed, amounting to 0.5 %. Specimens were re-weighed after drying. No measurable change in mass was then noticed. The individual results from flexural tests performed on these materials are presented in table 4.4.1 (no standard deviations are given since only five specimens could be exposed at a time).

No.	Control specimens	Exposed specimens
1	2030	1510
2	1980	1400
3	1770	1520
4	1840	1440
5	1760	1420
Ave.	1870	1460

Table 4.4.1. Strength loss of specimens immersed in water for seven months.
(All strengths are given in Newtons).

A dramatic loss of strength can be noticed here. In addition, a significant drop in modulus is observed during testing of exposed specimens. The drop in average strength amounts to 21 %. All exposed specimens (although exhibiting no measurable weight change) appeared lighter in colour than unexposed specimens. Voids on the material surface were larger than they were prior to immersion in water.

4.4.2 Gasoline

Specimens removed from gasoline baths after one, two, and three weeks were examined for signs of deterioration. A small increase in mass (average 0.3 %) was observed immediately after specimen removal. However, all specimens quickly dried and returned to their original masses. They did appear lighter in colour than the unexposed specimens (markedly so in some cases). Average strengths after different gasoline exposure times are plotted in figure 4.4.2.

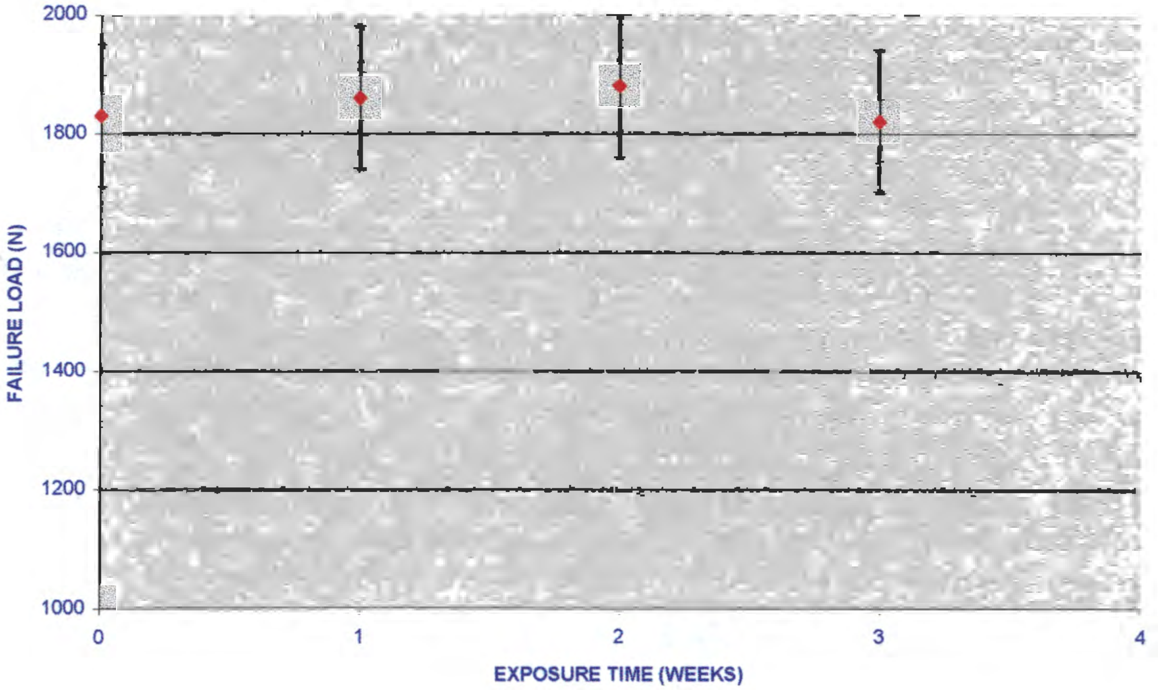


Figure 4.4.2. Graph of average failure loads vs time of exposure for gasoline immersion tests.

The results from this test indicate that no significant drop in strength can be expected after gasoline immersion for these periods. As with the first water test, the scatter of individual test results is larger than the variation in the averages plotted in figure 4.4.2. No prolonged gasoline exposure test is considered necessary.

4.4.3 Diesel fuel and engine oils

Specimens were immersed in commercial diesel fuel and motor engine oil for the same periods as for the other solvent tests. On removal, they were examined and weighed. Due to the oily nature and higher viscosity of these substances compared to gasoline, specimens remained impregnated with the oils for up to five days after removal from the immersion baths. A mass increase of up to 1 % was noted due to this effect. Apart from the darker appearance of the specimens due to impregnation of the diesel fuel and engine oils, no change of colour was observed.

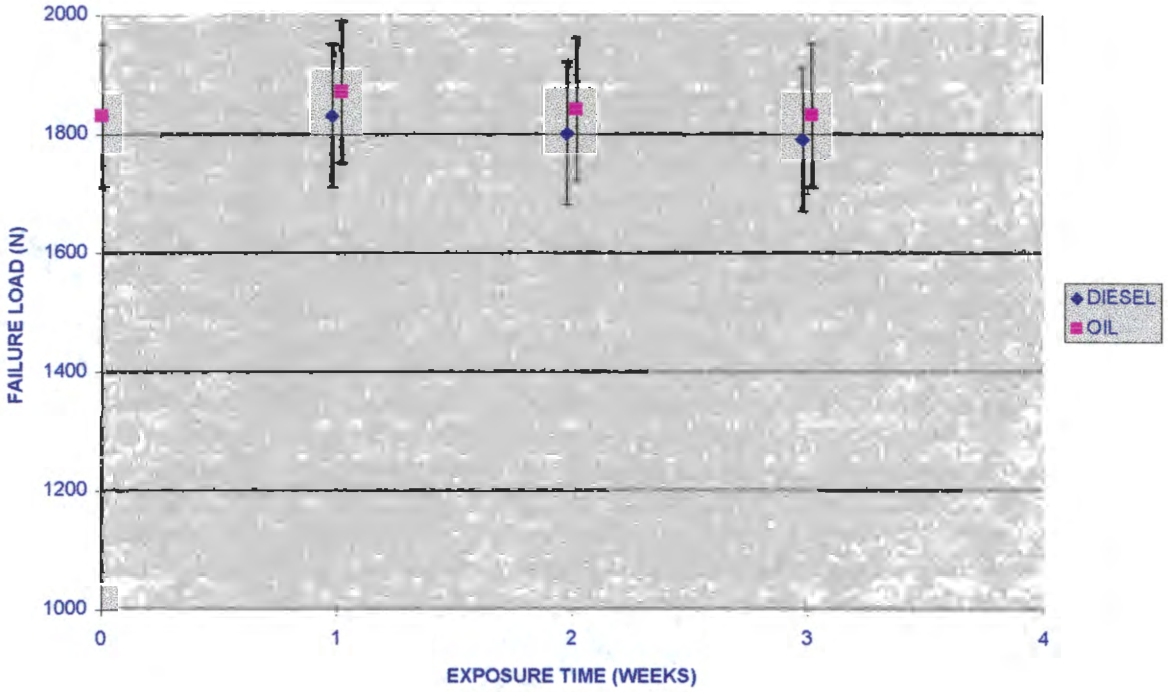


Figure 4.4.3. Graph of average failure loads vs exposure times for diesel fuel and engine oil immersion tests.

The above graph serves to indicate that no significant decrease in strength can be expected after immersion of polymer concrete specimens in diesel fuel or engine oil for these periods. In addition, there is no observed change in the *mode* of failure of the specimens during testing.

4.4.4 Saline solution

Strictly a sodium chloride solution in water should not be considered as a solvent. However, it is included in this section with other solvents since the same testing method and common control specimens were employed.

On removal from the saline baths specimens were visually examined for signs of deterioration. A small weight increase (0.5 %) was observed on initial removal, but after drying the specimens returned to their original masses. No visible sign of degradation could be detected after any of the exposure times.

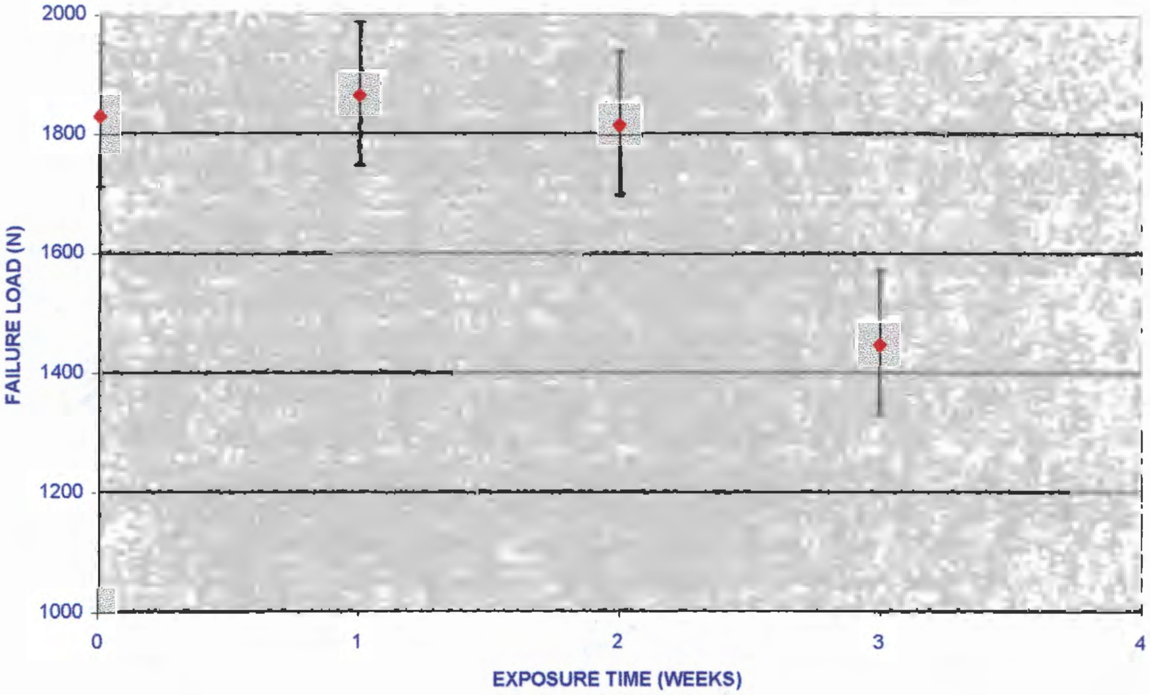


Figure 4.4.4. Graph of average failure loads vs time of exposure for saline solution immersion tests.

A considerable drop in strength after a three week exposure time can be observed from figure 4.4.4. This drop in strength amounts to nearly 20 %. In addition, the flexural modulus of the specimens appears to be diminished. It is evident that deterioration of mechanical properties of polymer concrete exposed to saline solution can occur.

In order to determine the effect of long term exposure of polymer concrete to a saline solution, specimens were immersed in the saline baths for a period of seven months.

On removal, visual inspection revealed that a noticeable discoloration of the specimens had set in. Initially black specimens appeared almost grey, and in some cases the aggregate and chopped fibres in the material had become visible through the specimen surface. No weight change could be detected after specimen drying.

The results of flexural tests performed on control and exposed specimens are presented in table 4.4.2 (no standard deviations are given since only five specimens were exposed at a time).

No.	Control specimens	Exposed specimens
1	1930	1330
2	1980	1190
3	1790	1210
4	1850	1040
5	1750	1240
Ave.	1860	1200

Table 4.4.2. Strength loss of specimens immersed in saline solution for seven months. (All strengths are given in Newtons).

The deterioration in mechanical strength of polymer concrete specimens immersed in saline solution for a prolonged period amounts to 35 %. Furthermore, a significant drop in the flexural modulus of the material was observed during testing. It is thus clear that severe degradation of polyester polymer concrete by saline solutions is possible over prolonged periods, and that the extent of the degradation depends on the saline concentration, and the length of time of continuous exposure.

To summarise: -

- Degradation by gasoline, diesel fuel, and engine oil does not occur to any significant extent over short exposure times.

- Exposure to water for short periods does not appear to significantly affect the mechanical strength of polyester polymer concrete. However, prolonged immersion in water can result in moderate to severe deterioration of mechanical properties amounting to a 20 – 30 % loss in flexural strength. Flexural rigidity may also be adversely affected.

- Exposure to saline solution can result in significant deterioration of mechanical properties over relatively short immersion periods. Severe degradation of polyester polymer concrete can occur over longer exposure times. The loss in flexural strength may be 35 – 40 %, and the flexural rigidity is also lessened.

4.5 Degradation by acids and alkalis

This section presents the results of tests involving the effect of acids and alkalis on the mechanical properties of polyester polymer concrete. The environments used were 10 % solutions of hydrochloric acid, sulphuric acid, and sodium hydroxide (alkaline solution). The pH levels of the acid and alkaline solutions were 1.5 and 13 respectively.

Specimens were removed from the aggressive environments at intervals of five, ten and fifteen days and examined for signs of degradation. Specimens immersed in sulphuric acid show visible deterioration, mainly loss of colour. Specimens immersed in hydrochloric acid and sodium hydroxide solution show more severe attack (loss of colour, yellowing, fibre and aggregate etching, attack of gelcoat, and enlargement of voids).

Specimens immersed in sulphuric acid show no measurable decrease in mass, while specimens immersed in hydrochloric acid in some cases exhibit a loss in mass of 0.4 % (particularly after the fifteen day exposure period). Immersion in sodium hydroxide solution results in a 0.5 – 1 % loss in specimen mass for the fifteen day exposure period.

Figure 4.5.1 presents the results of flexural tests conducted on polyester polymer concrete specimens after various exposure times. It should immediately be noticed that severe deterioration in mechanical properties occurs during exposure to these aggressive environments. Exposure to sulphuric acid for fifteen days results in an average drop in flexural strength of nearly 20 %. Hydrochloric acid produces an average decrease of 26 %, and sodium hydroxide solution 38 %. Although there is a significant amount of statistical variation in these values, it is quite clear that polyester polymer concrete does not perform well in acidic and alkaline environments. In particular, *alkaline* environments produce the most severe deterioration in flexural strength of this material. This result is consistent with the results of previous research regarding the subject.

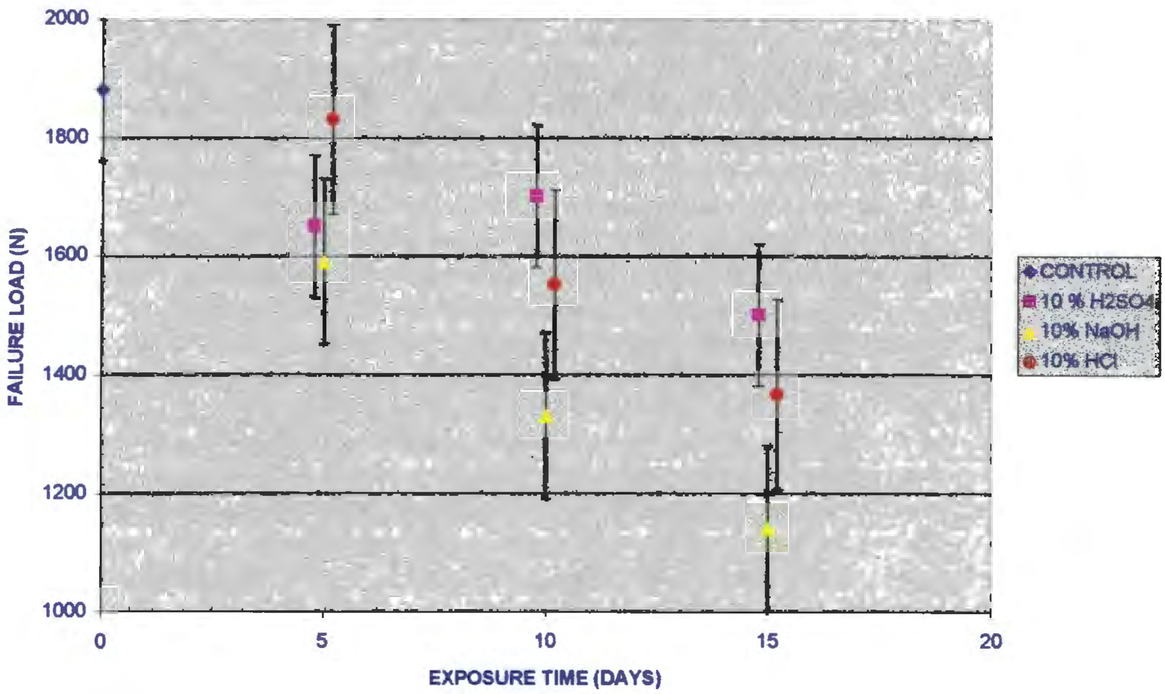


Figure 4.5.1. Graph of average failure loads vs time of exposure for specimens immersed in acidic and alkaline environments.

In addition to flexural strength loss, larger than usual *deviations* in strength were recorded during these tests (points in figure 4.5.1 have been separated slightly so that error bars can be distinguished). Standard deviations of 150 – 200 N are noted in some cases, and for this reason it is difficult to ensure reproducibility of these results. It is absolutely clear that severe deterioration of mechanical properties is possible. However, the *extent* of this degradation is subject to a degree of uncertainty due to the combined effects of the inherent variation in strength of the material and the nature of the attack by aggressive environments on a particular specimen.

The deviation in maximum failure loads for specimens is not the only variation observed. Complicated flexural test curves were obtained in some cases (see figure 4.5.2), and arbitrary variations in modulus and deflection were also recorded. Most of these fluctuations may be attributed to the chopped strand fibres, which are preferentially attacked. It seems that variation in the amount and nature of attack of fibre reinforced polyester polymer concrete by acids and alkalis is inherent to the material, and is thus to be expected.

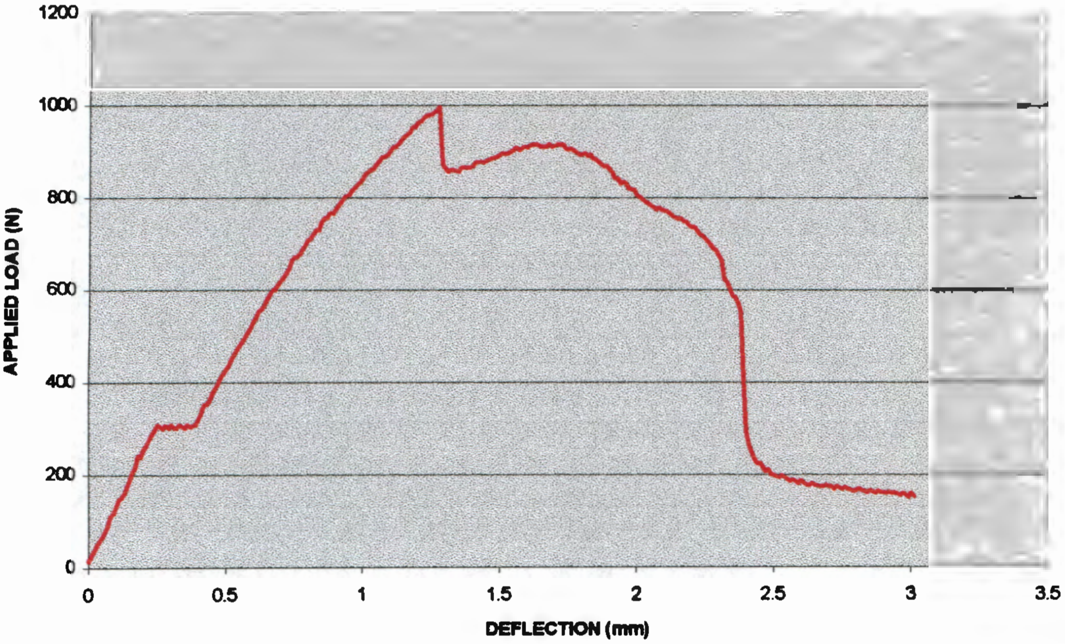


Figure 4.5.2. Example of chaotic flexural test curve obtained from specimen immersed in a 10 % sodium hydroxide solution for fifteen days.

Full discussion regarding the mechanism and significance of degradation by solvents, acids and alkalis is given in section 5.9.

4.6. Polymer concrete with graded aggregates

This section presents the results obtained from aggregate grading experiments that were performed with polyester polymer concrete. All work was conducted with a view to enable the resin content of the wet mix to be reduced. All mixes were made up using standard mix proportions of resin, aggregate, pigment, and chopped strand fibre reinforcement. However, the size distribution of particles making up the aggregate was varied.

4.6.1 Flexural strength and toughness

Groups of ten specimens for each mix variation were tested in flexure. The results from these tests are summarised in table 4.6.1. Included in this table are the proportions of each aggregate used for the mixes, and the standard mix currently used is presented for comparison.

	No. 2	No. 1	4/10	7/16	5-6.7mm	Ave strength	STD
Standard		100				1850	120
Mix 1	11.5	14		74.5		1180	130
Mix 2	14	74.5		11.5		1910	150
Mix 3	33	33		33		1910	150
Mix 4	25	25		50		1750	170
Mix 5	15	20	20	30	15	1300	180
Mix 6	15	40	15	15	15	1570	160
Mix 7	50	50				1790	200
Mix 8	10	45		45		1730	190

Table 4.6.1. Average flexural strengths of specimens made from aggregate graded polyester polymer concrete. All strengths given are failure loads in Newtons.

All proportions given in the above table are in weight percent, and all flexural strength and deviation values are given in Newtons. It should be noticed that specimens made from Mixes 1,5, and 6 all display flexural strengths well below that of standard polymer concrete. Specimens made from Mixes 2 and 3 exhibit a slightly higher flexural strength than standard specimens, however it is doubtful whether this is significant or not. Specimens made from Mixes 4, 7, and 8 all exhibit slightly lower flexural strengths than standard specimens. In general it appears that the addition of coarse particles to the mix has a deleterious effect on the mechanical strength of polymer concrete specimens. Further discussion on this issue and a final assessment of the viability of graded aggregates is presented in section 5.10.

The toughness of the flexural test specimens also seems to be deleteriously affected by the addition of coarse particles. Fracture toughness experiments were not conducted on graded aggregate specimens, however the behaviour of the material could be visually assessed. In the majority of cases where large aggregate particles are incorporated into the mix, failure of the specimens at critical load is complete. Where before there had been a certain amount of residual strength after specimen failure, there was now complete failure of the specimens. In particular, mixes having high void contents such as Mix 1, seem to exhibit low toughness characteristics.

Examination of the fractured specimens confirms that internally there are still a significant amount of voids in the material. Aggregate particles can be picked out of the matrix as if there is very little adhesion between them and the resin. In addition chopped fibres in the material can also be plucked out of the specimens. It is thus suspected that the reason for the poor performance of these specimens compared to standard specimens is that there was insufficient compaction of the wet mix in the moulds during manufacture of the specimens. It can be concluded that polymer concrete using large aggregates requires a better degree of compaction than polymer concrete using finer aggregates.

It must be remembered that in theory, correctly graded aggregates produce polymer concrete with low void contents and should thus improve the measured strength.

4.6.2 Workability

The workability of the wet mixes was visually assessed during the coarse of the moulding procedures. It was found that Mixes 1, 5, and 6 are very difficult to work with. The process of moulding specimens from these mixes is more time consuming than usual, since extra care has to be taken to ensure that the moulds are properly filled. All other mixes are reasonably easy to work with, although Mix 7 (with a high fines content) has a putty-like consistency that needs to be carefully packed into the moulds. It was suspected at this stage that vibratory moulding methods would improve the compaction achieved in the moulds.

4.6.3 Photographs of specimens

Arbitrary specimens from each mix group were selected for further analysis. They were sectioned and viewed using stereo optical microscopy in 'as cut' condition. The following figures show the microstructure of specimens made from seven of the eight different mixes and that of standard mix at 9X magnification.

Figure 4.6.1. Mix 1

Figure 4.6.2. Mix 2

Figure 4.6.3. Mix 3

Figure 4.6.4. Mix 4

Figure 4.6.5. Mix 5

Figure 4.6.6. Mix 6

Figure 4.6.7. Mix 7

Figure 4.6.8. Standard mix

The magnification used in all of these photographs is 9X. The resin matrix and the aggregates can be clearly distinguished. Important points noted are the mean free path through the material, the particle shapes, and the number of voids in the material. The micrographs showing Mix 1, 5, and 6 reveal the angular nature of the filter grits used in these mixes. Figure 4.6.7 shows up a number of voids that were present in the material. Voids can also be observed in some other mixes as well. It seems that voids cannot be avoided merely by using a (correctly) graded aggregate. Proper compaction and moulding methods are required to achieve any improvement in strength and possibly a reduction in resin content of the mix.

4.7 Effect of silane coupling agents

The results from flexural tests on specimens made from silane treated polymer concrete mixes reveal that this addition certainly improves the strength characteristics of the material. The improvement in most cases is substantial, but as before, seems to be subject to considerable variation. The table below presents the results of preliminary tests. In the first case, the silane was added directly to the *resin*, while in the second, the silane was distributed through the *aggregate* prior to addition to the resin. In both cases approximately 1 – 2 % silane (by weight of resin) was used.

No.	Standard	Silane treated resin	Silane treated aggregates
1	1820	2390	1920
2	1910	2110	2360
3	1650	2340	2120
4	1930	1780	1970
5	1900	2320	2000
6	1680	1970	2190
7	2050	2260	2150
8	1740	2080	2230
9	1880	2090	2060
10	1770	2530	2090
Ave.	1835	2190	2110
STD	125	220	140

Table 4.7.1. Flexural strength of standard mix compared to mixes containing silane additions. All strengths given here are failure loads in Newtons.

As can be seen from these results, the addition of silane coupling agent significantly improves the flexural strength of polymer concrete specimens. However, the deviations in these values are quite significant. Further tests were performed to establish how the *amount* of silane added to the mix affects the strength improvements.

Results of individual specimen tests are presented in table 4.7.1 in order to show the extent of variation in results. The strength readings show variation that is considered typical of polymer concrete. The following table shows *average* flexural strengths and deviations for specimens made up from mixes containing varying amounts of silane coupling agent. In this case the silane was added directly to the polymer concrete mix, rather than to the resin or the aggregates.

% Silane	Ave. strength	STD
0.25	1960	180
0.5	1960	140
1	2100	150
2	2000	230
5	2010	200

Table 4.7.2. The effect on strength of variation in the amount of silane added to a polymer concrete mix. Strengths given are failure loads in Newtons.

The results presented in table 4.7.2 are plotted in graphic format in figure 4.7.1.

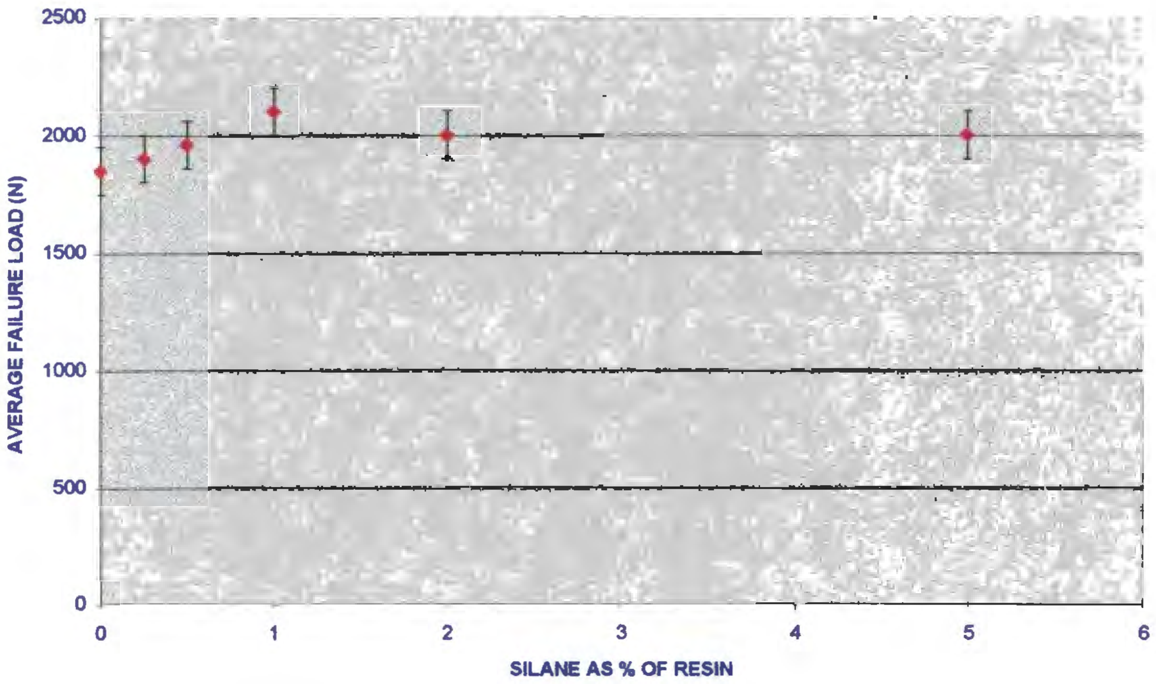


Figure 4.7.1. Graph of average flexural strength vs amount of silane added to mix.

Although the standard deviations overlap substantially in figure 4.7.1, a trend can be observed. The evidence suggests that there is no benefit in adding more than 1 % silane to a polymer concrete mix. Also, silane additions as low as 0.25 % still seem to improve the strength of polymer concrete.

What might also be noticed, is the fact that the strength gains achieved in this experiment are not as great as those for the preliminary tests conducted. It is suspected that the method of silane addition to the mix is the reason for this. Better strength improvements are achievable by mixing the silane with the resin or the aggregates rather than by adding it directly to ready mixed polymer concrete.

Specimens with silane additions not only exhibit higher flexure strengths than usual, but they appear to be tougher as well. K_{IC} for silane treated polymer concrete is of the order of 1.98 $MNm^{-3/2}$, a slight improvement on non treated polymer concrete (1.85 $MNm^{-3/2}$).

The nature of the wet mixes was visually assessed during the course of the work. It was noticed that the mixes made using a silane coupling agent are 'wetter' than usual, and are thus more workable. The silane added has an oily type of feel that is easily noticed in the polymer concrete mixes even at low percentage additions. The train of thought that arises from this, is that it might allow the actual resin content of the mix to be reduced, whilst retaining the same original workability. Further discussion of these results and potential ideas are presented in section 5.11.

4.8. Effect of postcuring procedures

Three sets of tests were carried out to determine the effects on strength of postcuring procedures. All postcuring was carried out at 80° C for three hours. The state of cure of polymer concrete is dependent not only on postcuring, but on other factors such as amount of catalyst and accelerator added. Minor variations in these values are possible from mix to mix, and so for comparison sake, ten specimens were made up from a mix, five were kept aside, and five were postcured (to ensure constant conditions).

The results obtained from preliminary tests are presented in table 4.8.1. No standard deviations are given since only five tests could be performed for each condition of the material

No.	Postcured	Non postcured
1	1820	1740
2	1850	1840
3	1600	2200
4	1930	1940
5	1900	1550
Ave.	1820	1850

Table 4.8.1. Effect on flexural strength of postcuring procedures. All strength values given are failure loads in Newtons.

These results show that postcuring procedures do not necessarily improve the mechanical properties of the material. However, the consistency of the results appears to be improved after postcuring. A second experiment was performed to verify these results. The same experimental method was used as for test 1, except that two separate batches of polymer concrete mix were tested.

	Batch 1		Batch 2	
No.	Postcured	Non postcured	Postcured	Non postcured
1	1720	1790	1760	2190
2	1800	1900	1840	2180
3	1620	2010	2010	1880
4	1880	1950	1930	1800
5	1740	1940	2070	1950
Ave	1750	1910	1920	2000

Table 4.8.2. Effects of postcuring for verification tests. All strength values given are failure loads in Newtons.

These were perhaps the most contradictory results obtained through the entire course of the research undertaken. It seems even non postcured specimens show greater than average strength (1850 N). The fact that postcuring does not improve the consistency of results in this test implies that the consistent results obtained in test 1 are probably more of a statistical phenomenon than anything else. Ideally, at least ten specimens in each condition should have been tested, but there were limitations to the number of specimens that could be made at a time.

A third test was designed to investigate the effect of postcuring. In this case two batches of ten specimens were made up (ten at a time), under carefully controlled conditions, ensuring equal mix characteristics, catalyst additions, mixing time, and moulding time. Specimens from each batch were then interchanged randomly, so that twenty specimens were attained each made under (as close as possible) the same conditions. Ten specimens were then postcured in the standard way, and ten specimens were kept aside for comparison.

No.	Postcured	Non postcured
1	1840	1790
2	1710	2000
3	1640	1900
4	2150	1520
5	1960	1840
6	1740	1640
7	1740	1890
8	1640	1800
9	1860	2060
10	1810	1940
Ave.	1810	1840
STD	165	160

Table 4.8.3. Third postcuring test results. All strength values are given in Newtons.

The results from this test confirm that postcuring procedures have no beneficial effect on the flexural strength of polymer concrete. It seems that the main reason that postcuring procedures are recommended by resin manufacturers is that the rigidity of the cured resin is improved. Postcuring affects the *state of cure* of the resin, which in turn affects the rigidity or modulus of the resin, rather than it's flexural strength.

Indeed, higher rigidity of the postcured polymer concrete specimens has been observed during testing. It should be mentioned that exact values of modulus cannot be quoted here. The problem is analagous to the fact that a tensile test curve should never be used to measure Young's Modulus for metals. Nevertheless, visual inspection of the flexural test curves confirms that the rigidity of postcured polymer concrete is higher than that of non-postcured polymer concrete.

4.9 Effect of vibration methods

Experiments conducted using vibration techniques involved four preliminary experiments. The first test involved the vibration of catalysed mix already placed in the mould. In the second test, the mix was vibrated *during* the moulding procedure. The third test was performed in the same way as test 2, however 1 % silane had been added directly to the mix prior to moulding. Test 4 involved the use of a layering technique. Silane treated mix was used to half fill the mould and allowed to settle for three minutes. The mould was then filled to the top (again during vibration).

No.	Test 1	Test 2	Test 3 (silane)	Test 4 (silane)
1	1480	1830	1800	1780
2	1380	1650	1930	1820
3	1930	1690	1910	1830
4	1430	1730	1590	1720
5	1800	1580	1900	1800
6	1660	1600	1680	2160
7	1400	1560	1830	2200
8	1550	1410	1720	1880
9	1570	1440	1590	2120
10	1350	1330	1950	1830
Ave.	1560	1580	1790	1920
STD	190	150	140	170

Table 4.9.1. Individual results from vibrated specimens. Strengths are given in Newtons.

The vibration time for test 1 was about five minutes. For test 2 and 3 the vibration time was about ten minutes, and for test 4, the vibration time was about 14 minutes. The results presented in Table 4.9.1 indicate that the type of vibration performed appears to be detrimental rather than beneficial to mechanical strength of polymer concrete. Comparison of the average flexural strengths in tests 2 and 3 confirms that silane additions to the mix improve the flexural strength of polymer concrete. It appears, however, that the vibration procedure has *decreased* the strength of the material substantially. In view of the fact that vibration methods were employed to rid the material of air bubbles and voids (to increase the strength), this result serves to indicate that either the experimental method was at fault, or that some other phenomenon was occurring that had not been anticipated.

It was suspected before these tests were conducted that *segregation* of the aggregate and resin might occur during vibration. The heavier aggregates settle to the bottom of the mould, while the resin collects on the top surface. The result in a hardened specimen is that the bottom surface is depleted in resinous binder, while the top surface becomes resin-rich. Another factor that was considered was that the width of the specimen moulds might be too small to allow air bubbles to escape through the viscous polymer concrete mix, ie. the mould walls restrict flow of the mix.

An assessment of the viability of employing this technique is presented in section 5.13.

5. DISCUSSION

This section presents an analysis of the experimental results obtained. It also includes an analysis of the validity of these results, and due to the high standard deviations obtained, details the methods used to draw relevant conclusions from these results. A full catalogue of AV Mouldings product problems or failures is also presented here.

5.1. Previous failures and problems

This section relates specifically to the manufacturing process that AV Mouldings is currently involved with. As detailed in section 2.5, AV Mouldings manufacture a variety of manhole and drain components from polymer concrete. As in any manufacturing process, careful control of conditions is required to ensure consistency of performance of products in service. Good control of ***material properties*** coupled with sensible ***design*** procedures results in reliable products that are compatible with service conditions. For this reason, the analysis of problems associated with AV Mouldings products falls into two categories. These are the material characteristics and the product design characteristics.

5.1.1 Material characteristics

Common material defects associated with polymer concrete are discussed in section 2.3.2. Table 5.1.1 lists all the material defects that were noticed during detailed examination of products. Since polymer concrete is a composite material, the number and type of possible defects are extensive. Each type of defect may affect polymer concrete products in different ways, to a greater or lesser degree, but it is generally accepted that *all* defects have a deleterious effect on the mechanical performance of polymer concrete.

For the reasons mentioned above, a full analysis into the effects of defects was conducted.

Type	Cause	Comments
Voids	Air bubbles introduced during mixing. Improper compaction in moulds	Main cause of failures attributed to material defects
Wet spots	Improper mixing of catalyst into mix	Surface wet spots easy to detect. Internal wet spots not generally critical
Undissociated chopped fibres	Insufficient mixing of resin/fibre mixture	Not generally critical. Can cause stress concentrations
Poor resin impregnation of woven matting	Mix too viscous. Insufficient time allowed for impregnation	Product severely weakened. Complete shear possible
Inconsistent resin-aggregate ratio	Mix allowed to segregate in storage drums. Not properly remixed prior to use	Not generally critical. Results in inconsistencies in strength
Chopped fibre 'clump' formation	Resin/fibre combination mixed for too long.	Introduces inconsistencies in strength
Foreign objects	Too little control of quality of mix additions	Produce stress concentration effects in products
Undercure	Too little catalyst added to mix Presence of water in mix.	Insufficient strength and rigidity may lead to premature failure
Poor resin-aggregate adhesion	Excessive shrinkage during cure. Too much catalyst added	Accelerated degradation possible

Table 5.1.1. Different types of material defects and problems in polymer concrete.

It has already been established that it is of the utmost importance to minimise the number of defects in a polymer concrete moulding (see section 2.3.2) to ensure optimum properties. Figures 5.1.1 (a) to (d) illustrate some of the more common defects observed during examination of products.

Figure 5.1.1 (a). Voids in materials.

Figure 5.1.1 (b). Chopped fibre 'clumps' in a polymer concrete moulding.

Figure 5.1.1 (c). Poor resin – mat adhesion (in this case accelerating failure).

Figure 5.1.1 (d). Undissociated chopped fibres.

The solutions to most of these problems can be arrived at by ensuring that the following procedures are adhered to: -

- Proper compaction and vibration of mix in moulds in order to minimise the number of voids in the hardened material.
- Thorough mixing of catalyst into mix to reduce chances of wet spots forming.
- Removal of *any* foreign particles from raw materials and final mix.
- Ensure enough time for resin to impregnate woven roving matting before application of subsequent layers of mix in moulds.
- Ensure that mix has the required consistency prior to moulding. If this is not the case, the mix should be thoroughly reworked to achieve this consistency.
- Ensure that the correct amount of catalyst is added to the wet mix to facilitate complete cure.
- Control of mixing time to ensure dissociation of chopped fibre strands without fibre 'clump' formation.
- Ensure that moisture (in any form) is not brought into contact with wet or semi-cured mix.

It is believed that most of the procedures outlined above can be performed quite simply, but some are not so easily achieved – in particular the issue of avoiding undissociated fibres and chopped fibre 'clump' formation during mixing. However, these problems are not considered to be critical. At worst, it is believed that minor stress concentrations can result from undissociated fibres, while inconsistencies in strength result from fibre 'clump' formation.

5.1.2 Failures due to design faults

Cast iron manhole covers and frames are generally overdesigned, and failures are accordingly very rare. However, polymer concrete manhole covers and frames manufactured according to cast iron specifications have exhibited a couple of failures in service. The reason for this is that there are considerable differences in mechanical properties between the two materials. It has been found that in the vast majority of cases, that cracking around the lip region of covers is the cause of premature failure. It seems that the cyclic nature of the loading condition on these covers in service causes cracks to be initiated in this region and they propagate through the cover after each successive applied load (analogous to fatigue in ductile materials). Figures 5.1.2 (a) to (d) illustrate the type of cracking that occurs, and the four photographs present successively more advanced cases of this cracking.

Figure 5.1.2 (a). Case 1

Figure 5.1.2 (b). Case 2

Figure 5.1.2 (c). Case 3

Figure 5.1.2 (d). Case 4

For Case 1, fine hair-line cracks are visible running parallel to the circumference of the cover in the lip region. In addition, surface voids can be clearly seen. In Case 2, larger cracks are visible. They are wider and longer than they were in Case 1 and a certain amount of crack branching can be seen. It appears that crack propagation is not completely limited to the radius of the lip region, since the cracks also run along the inside surfaces of the lip and bulk sections. The cracks depicted in Case 3 are in a highly advanced state. They run completely around the circumference of the lip region, and have propagated through to the top surface of the cover in some areas. This case can be taken as the critical condition for failure. The cover in Case 4 has completely failed. The component illustrated is the bulk section that catastrophically sheared from the supporting lip section. Undissociated chopped fibres are clearly visible in figure 5.1.2 (d) as well.

Figure 5.1.3. Crack propagation through to top surface of cover.

The above picture illustrates an advanced case of lip cracking. Here, the cracks have propagated completely through to the top surface of the cover. In general, they follow a path along the surface coinciding with the lip circumference, ie. in circular formations a short distance in from the edge of the cover. A certain amount of crack branching is visible. This particular cover had not yet fractured, but it is clear that the extent of cracking in this case has become critical.

Figure 5.1.4 (a). Photograph showing the plane of fracture through reinforcement.

Figure 5.1.4 (b). Fractured component

The photographs in figure 5.1.4 confirm the fact that woven roving reinforcement can strengthen a product, but after the initiation of cracking, it can also act as a weak point along which crack propagation and shear can take place. Figure 5.1.4 (b) shows a cover that had completely fractured. The lip section (on the right) contains the remains of the woven roving reinforcement on its top surface. Figure 5.1.4 (a) is a closer view of the same surface. It is clear in this picture that failure had occurred along the plane of the woven roving reinforcement. This picture also illustrates cases of poor resin/mat impregnation and undissociated chopped fibres.

5.2 Theoretical failure modes due to design faults

5.2.1 Failure by fast fracture due to critical crack growth

The following discussion presents the basic theory of failure of these covers. It should be remembered that this type of failure is by far the most common fracture mechanism. Other failures can usually be attributed to faulty workmanship, or material manufacture, not to an inherent design problem.

It is believed that cracks originate in the lip region due to the high bending stress associated with the thinner lip section. Crack formation is accelerated by the stress concentrating effects of the small radius of curvature employed in the design. The actual initiation points for crack formation are very likely to be at discontinuities in the material, eg. voids and cavities, foreign objects embedded in the material, etc.

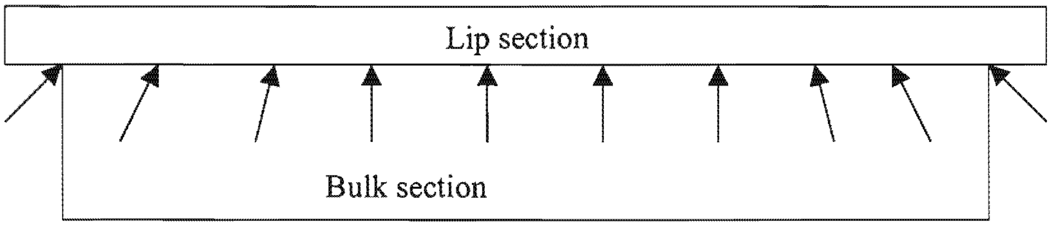


Figure 5.2.1. Schematic showing the extent of crack initiation points.

The load imposed on a manhole cover in its frame causes it to bend slightly. It must be remembered that the cover can be considered to be simply supported around its edge under this loading condition. The edges are restrained from lateral movement due to the side of the frame, but they are not clamped or rigidly held in position. Essentially the cover consists of two parts, the bulk section, and the lip section, and in general, the thickness of the lip section is much less than that of the bulk section. Due to the thickness of the bulk section, it is far more resistant to deflection caused by bending stresses. The lip section can theoretically deflect more than the bulk section, but it is also restrained from this deflection since it is connected to the bulk section. The combination of the bending stresses and the deflection constraint result in the following deflection tendency: -

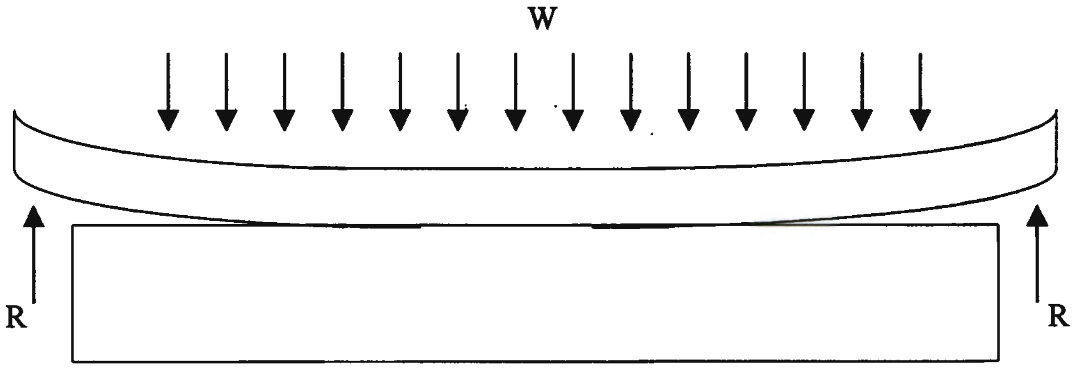


Figure 5.2.2. The 'peeling' tendency associated with bending stress imposed on a Type 2A manhole cover (in two dimensions).

The schematic presented in figure 5.2.2 is an exaggerated view of the deflection tendencies associated with an applied bending moment (shown as arrows in figure 5.2.2). It is clear that the lip section tends to deflect more than the bulk section, resulting in high stress concentrations at the corners under the lip.

The type of situation illustrated in figure 5.2.2 is the deflection that the cover would undergo if it *were not restrained* from doing so. The fact that the two sections are connected, infers that one section cannot deflect more than the other. They are thus forced to deflect in a mutual manner under equilibrium laws. It is this restraining force that results in the stresses needed to initiate cracks. The circumferential stress concentration under the lip in the corners is increased further due to the small radius of curvature there. Cracks initiate at local areas of even higher stress, such as voids in the material. After each cycle of loading, the cracks propagate a small distance further into the material along preferred paths of low energy.

It is clear from figure 5.2.2 that the direction of growth of cracks through the material will be at some angle to the normal direction, rather than in the normal direction. If there was no bending deflection, then shear forces would be dominant, and crack propagation would be essentially vertical, through the lip. However, in this case bending *does* occur, and crack growth at an angle through the lip is to be expected.

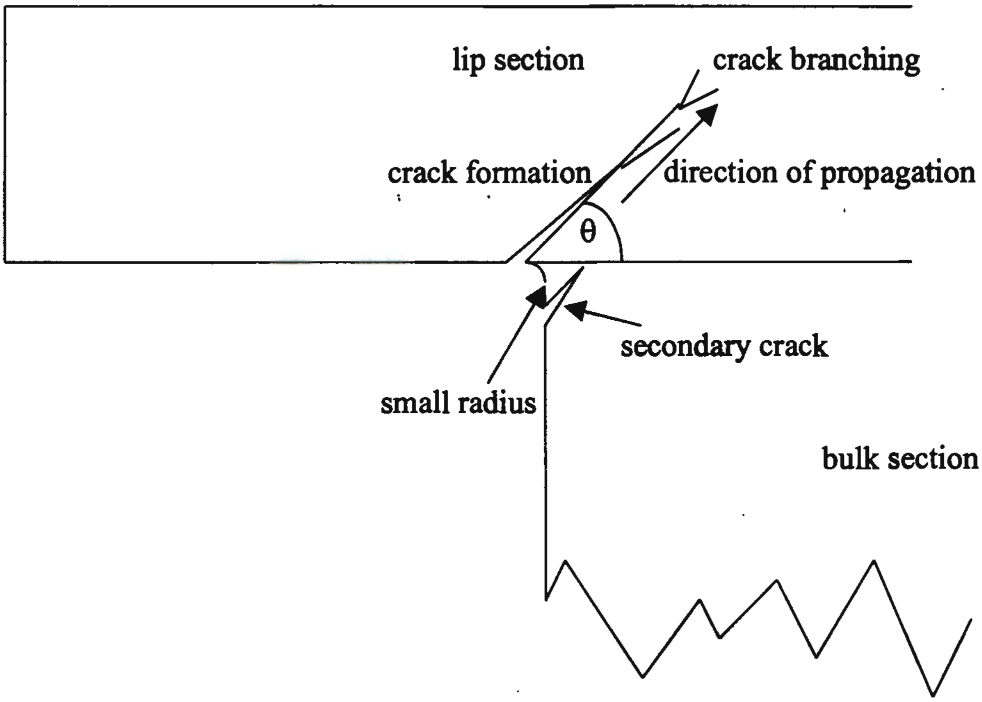


Figure 5.2.3. Diagram illustrating the formation and propagation of cracks in the lip region of a cover.

The angle θ, in figure 5.2.3 is dependent on the different response to bending of the two sections. Cracks can initiate at any preferred sites around the circumference, and they propagate slowly by fatigue processes through the thickness of the lip as illustrated. In certain areas around the circumference of the cover, they can propagate completely through the lip thickness. This may or may not be the critical case for catastrophic failure, since the remaining ligament sections may still possess sufficient strength to prevent this occurrence. However, it is clear that failure in this condition is inevitable.

Catastrophic failure will occur when there are not enough areas of contact to support any further service loads. On application of further loads, existing cracks that have become critical extend catastrophically through any remaining ligaments holding the component together, and severence occurs by fast shear. The bulk section then falls into the manhole, and the lip section generally remains as a rim around the edge of the manhole opening.

5.2.2 Failure along preferred paths

The theory of failure along preferred paths follows the same theory as previously discussed. The mechanisms are identical, however in some cases cracks encounter paths that make their progress through the material easier. In most fracture cases that were observed, woven roving reinforcement was placed in a position through the thickness of the cover that was in the line path of crack growth. This reinforcement is known to strengthen polymer concrete considerably, however it also acts as a path for crack growth under cyclic loading conditions.

Figure 5.2.4 is a schematic illustrating crack growth along the plane of the woven roving reinforcement. The mechanism by which the reinforcement becomes a preferred path for crack growth is related to the energy required to create new surfaces. It is believed that the energy required to separate the reinforcement from the surrounding material is less than that required to create new surfaces within the bulk material. For this reason cracks require less energy to propagate along the reinforcement/bulk material interface than they do to propagate through the bulk material itself. This problem is compounded by the fact that in many cases there is poor impregnation of the matting by the resin, resulting in poor interfacial bond strength in these areas.

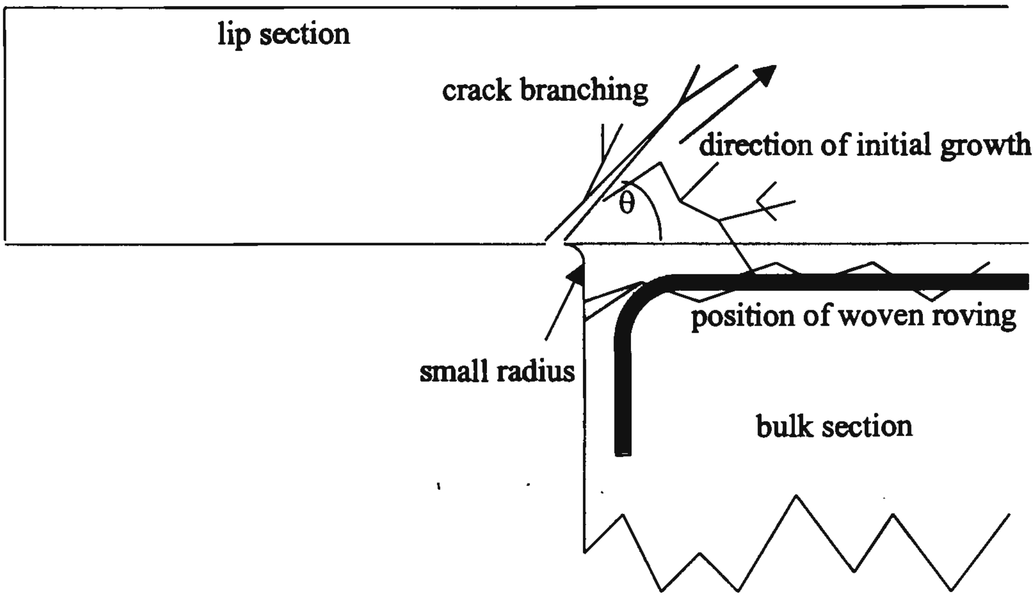


Figure 5.2.4. Schematic showing the propagation of cracks along a path of low interfacial bond energy.

Although the initial direction of crack growth is similar to that shown in figure 5.2.3, the chances of crack branches 'discovering' the easy path provided by the woven roving are high. Cracks then preferentially follow this path. In some cases the main cracks may propagate through to the top surface of the cover, in addition to growth along the preferred path. The overall result is a highly weakened component.

Catastrophic failure occurs when crack growth has reached a critical stage. In this condition the remaining ligaments cannot hold the component together under further applied service loads. Fast shear will occur along the preferred path, severing the remaining ligaments. As before, the bulk section falls into the manhole. The lip section remains essentially in one piece and may still cover the manhole opening.

Indeed, not only woven roving is responsible for this type of failure. *Any* discontinuity (eg. lifting slot keyholes) may provide a preferred path for crack propagation.

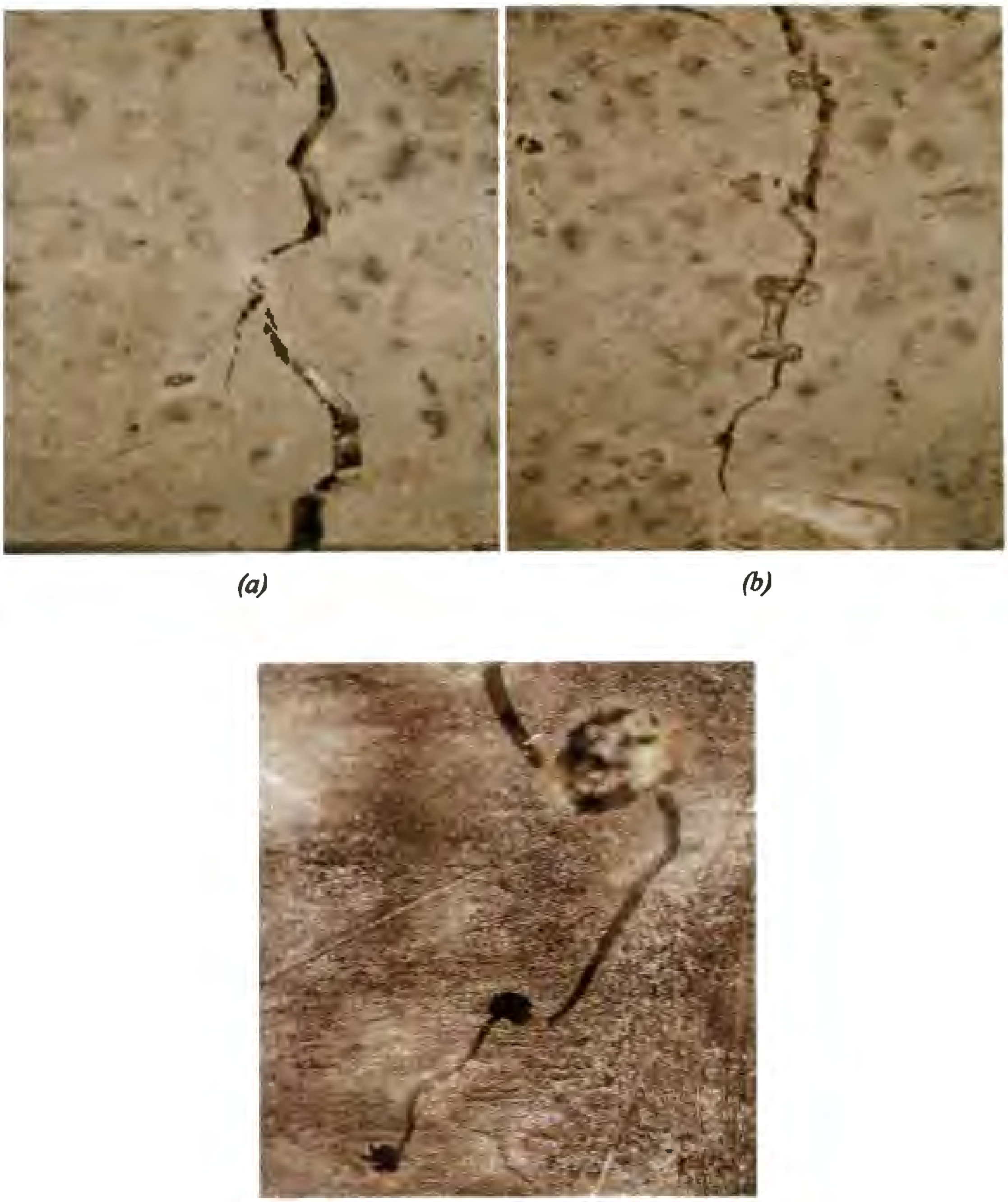
To summarise :-

- Bending moments during loading result in stresses being developed along the circumference of the lip/bulk intersection.
- The small radius in the corner produces high stress concentrations.
- Cracks initiate from local stress concentrations such as voids and other discontinuities in the corner region.
- Cracks grow by processes similar to *fatigue* during each cycle of loading.
- Cracks follow preferred paths through the component eg. reinforcement or lifting slot/bulk material interfaces.
- Catastrophic failure occurs when the remaining ligaments cannot support further applied loads.

5.3 Failure mechanisms in polymer concrete

5.3.1 Crack propagation

This discussion is based on examination of fractured specimens taken from the various flexural tests performed to characterise polymer concrete. Figures 5.3.1 (a), (b), and (c) present photographs of standard flexural test specimens after failure.



(a) *(b)*

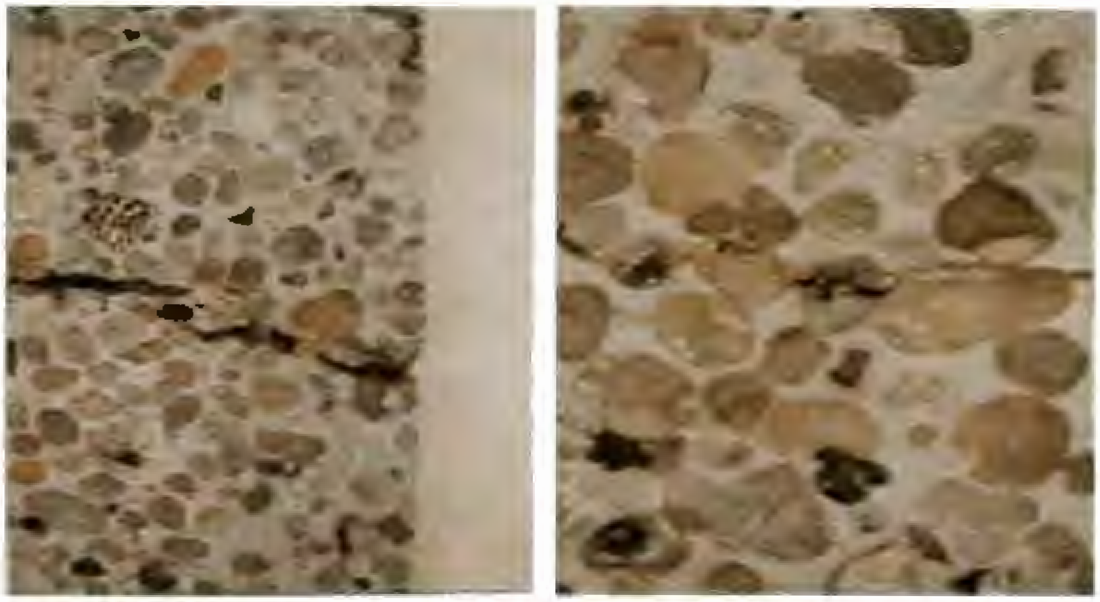
(c)

Figure 5.3.1 (a), (b) and (c). Photomicrographs of fractured flexural test specimens. (a) and (b) at 9X magnification, (c) at 55X magnification.

Figure 5.3.1 (a) is a magnified view of a fractured specimen, showing the crack path along the *surface* of the material. Crack branching can be clearly seen. In addition, the fractured ends of the chopped fibre reinforcement in these specimens are evidence that the propagation of the crack requires the pullout, and in some cases fracture of the glass fibre reinforcement. It is thus clear that the fibre reinforcement must significantly toughen the material (as already established in section 4.1.5). The photographs in figures 5.3.1 (b) and (c) show how cracks follow a path of lowest energy. In this case voids on the surface form a path of lowest energy for crack propagation. Although these pictures illustrate surface phenomena, it must be assumed that the same mechanisms are prevalent throughout the material (voids and fibres are not found exclusively on the material surface).

Cracks propagate through a material along paths of *lowest energy*. Polymer concrete consists mainly of a resin matrix with a mineral aggregate filler and the interface between these two components are essentially bonded surfaces. The propagation of cracks requires the creation of new surfaces, which have an associated energy. For this reason a crack preferentially follows a path that requires minimum creation of new surfaces. In this case cracks follow (as closely as possible) a path through the material along the interfacial regions of the resin and the aggregate particles. This behaviour is confirmed in the photomicrograph shown in figure 5.3.2 (a). The photograph in figure 5.3.2 (b) illustrates the fact that although the *majority* of crack propagation is between the aggregate and resin interface, in certain cases cracks may propagate through the aggregate particles themselves.

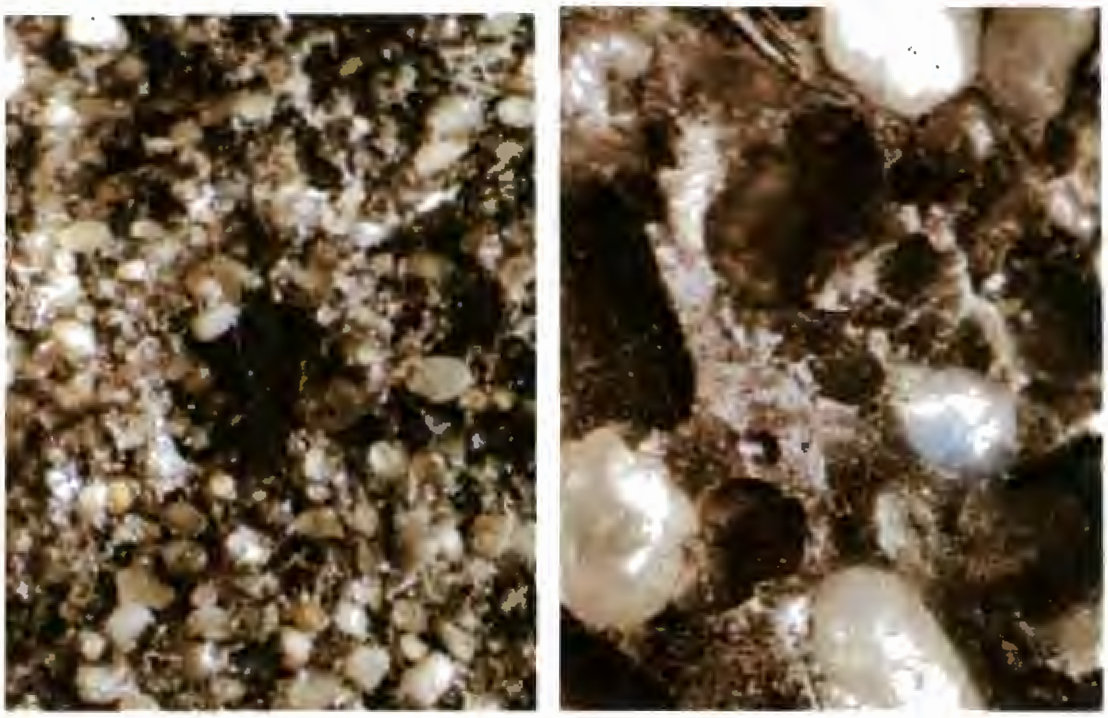
As a result of the facts so far discussed, it seems reasonable to deduce that the resistance to crack propagation (toughness) of polymer concrete is related to the bonding energy of the resin/aggregate interface. The added effect of the chopped fibre reinforcement further toughens the material. In this case the added toughness is obtained due to the energy required to pull the fibres out of the surrounding matrix. The energy required to achieve this is in turn related to the bond strength of the resin/fibre interface. This reasoning explains how the improved toughness of polymer concrete made from silane treated aggregates is achieved. In essence, the silane acts as a coupling agent, improving the bond strength between the aggregates and the resin (see section 5.11).



(a) *(b)*

Figure 5.3.2 (a) and (b). Photomicrographs showing the preferred path of crack propagation through polymer concrete. (a) at 9X magnification, (b) at 20X magnification

The photomicrographs below depict the *fracture surface* of standard polymer concrete taken from the beam specimens used in the investigation.



(a) *(b)*

Figure 5.3.3 (a). Fracture surface of standard polymer concrete at 9 X magnification. (b). Part of the same surface at 55 X magnification.

Large voids are clearly seen in figure 5.3.3 (a). In addition, the interweaving chopped fibres are also visible. In (b) the resinous matrix appears as the dark cratered material. The craters represent areas where the aggregate particles were pulled out of the material during fracture. Some glass fibres are also visible at the top of the picture. The evidence confirms the theory that crack propagation progresses through the resinous matrix, generally around the aggregate particles, and results in debonding of the particles from the matrix.

To summarise: -

- Polymer concrete is essentially a *brittle* material that fails due to cleavage rather than ductile yielding. Some local yielding of the resin matrix may occur, but is limited by the restraining effects of the aggregate filler.
- High internal stress is produced when a load is applied. Most of the deflection measured is due to the elastic behaviour of the resinous matrix.
- Since yielding is restrained by the aggregate filler, the material cannot significantly relieve the internal stress by undergoing deformation. At higher loads the internal stress may become high enough to break the interfacial bonding between the aggregates and the resin matrix. These areas can act as crack initiation points.
- In practice, this condition is not usually reached, since voids in the material become crack initiation points at lower stress levels than those required to break the interfacial bonds.
- ***Once initiated, the stress required to propagate cracks is vastly reduced.*** The flexural curve shown in figure 4.1.1 on page 49 confirms this behaviour. Indeed, most of the residual strength the material has is due to the chopped glass fibre reinforcement.
- Failure under cyclic loads occurs by a process similar to fatigue. Each loading cycle causes the crack to grow slightly until it reaches critical length. Fracture then occurs by fast brittle cleavage. The path of the cleavage plane follows the interfacial boundaries of the resin matrix and the aggregate filler particles.

5.3.2 Case study of fractured component

A full investigation has been conducted on four Type 2A replacement covers that had exhibited lip cracks due to demanding service conditions. The road in question is located in an industrial area where heavy vehicle traffic is common. A sample of the effluent running through the drainage system has been analysed and found to be a sugary substance of pH 12 at a temperature of about 40° C. It had condensed on the underside surfaces of the manhole covers in the road. The following sequence of events is the cause of cracking in these covers: -

1) Condensation of the aggressive alkaline effluent on the underside of the covers and in particular in the lip regions of the covers led to degradation of the mechanical strength of the material. Seepage into surface voids followed by attack of the interfacial bonds between the resin matrix, aggregates, and the glass fibre reinforcement resulted in substantially weakened components.
2) Repetitive heavy traffic flow resulted in high loading of the already weakened components. The small radius under the lip section coupled with the high strain rates associated with traffic flow magnified the stress in these areas of the covers.
3) Voids on the surface of the material in the lip regions further increased the stress concentration above the fracture stress of the weakened components.
4) Cracks initiated at some preferred point of highest local stress in the lip regions. These cracks quickly propagated around the circumference of the components.
5) Once initiated the stresses required for crack growth into the material were much reduced, and resulted in accelerated propagation of the cracks.
6) Cracks grew through the lip section in some sites to the top surface of the covers, and in others they followed the plane of the woven roving reinforcement.

Fortunately the problem was detected prior to complete failure of the components, and they were subsequently replaced. However, the replacements are not expected to perform any better than their predecessors, and it is thus suggested that a more permanent solution is required.

5.4. Statistical methods

The methods used to assess the validity of experimental results are discussed here. As previously established, the variance in individual test results is significant in most cases. It is believed that the cause of these high variances is due to the inherent properties of fibre reinforced polymer concrete. It has been established in section 4.3 that polymer concrete without fibre reinforcement does not display the same high variances. It is these variances that complicate the deduction of relevant conclusions from the results.

5.4.1 Confidence intervals

The statistical method of confidence intervals is used to analyse most of the results obtained. This method involves expressing a certain confidence that a value falls within a specified range. If a sample of size n, is drawn from a population of size N, then confidence intervals may be used to express how the mean of the sample may be related to the mean of the entire population. The mean and standard deviation of the sample are usually denoted by $\bar{x}$ and s. The mean and standard deviation of the population as a whole are usually denoted by μ and σ. An interval can be calculated that contains μ with a certain degree of confidence based on the calculation of $\bar{x}$ and s from the sample. The calculation of this interval assumes that the population is normal (or shows no large deviations from a normal distribution). Figure 5.4.1 is a plot of a typical normal distribution. A further approximation used is that the standard deviation of the sample, s, is equivalent to the standard deviation of the entire population, σ. Strictly this may not be entirely correct. However, since the standard deviation of the population is not known in most cases, the aforementioned approximation is made.

The calculation of confidence intervals relies on probability density curves related to the normal distribution curve. Various statistical tests have been performed to assess whether the results obtained could be approximated by a normal distribution curve. In some isolated cases the Weibull distribution curve seems to fit the data more accurately, but overall the readings show no large deviations from the normal distribution curve.

After establishing the 'normalness' of the data obtained, a desired confidence is chosen for which an interval can be calculated that contains μ (with that chosen confidence level).

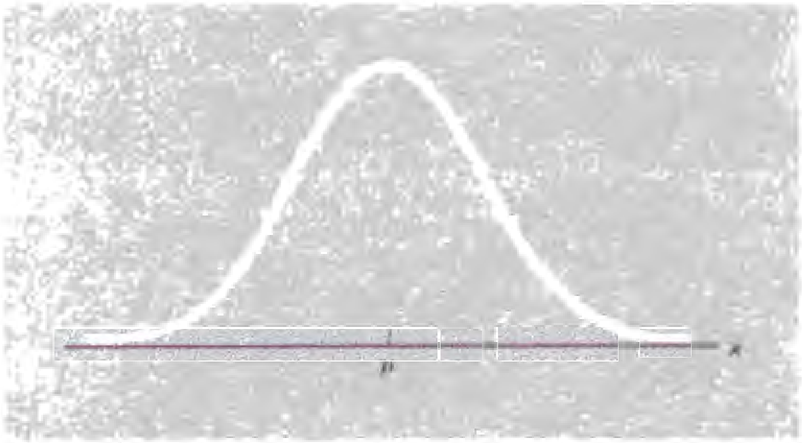


Figure 5.4.1. Plot of a typical normal distribution[15].

Confidence intervals are calculated using the following equation: -

$$\bar{x} - t_{\alpha/2}\cdot\frac{s}{\sqrt{n}} < \mu < \bar{x} + t_{\alpha/2}\cdot\frac{s}{\sqrt{n}} \tag{5.1}$$

This equation is generally valid for small samples from a population ($n < 30$). $t_{\alpha/2}$ is a number depending on the type of normal distribution, size of the sample taken, and degree of confidence desired. For instance, if the sample size, n, is 20, and the required confidence level is 99 %, then statistical tables give $t_{\alpha/2}$ to be 2.861. Using the sample size, n, and the measured values of $\bar{x}$ and s, an interval can be calculated that has a 99 % probability of containing μ.

5.4.2 Application to experimental results

The work conducted in this research involved the testing of sample specimens from different manufacture, treatment, and exposure conditions. It is desired to know with how much confidence the true mean of the population from which the sample was drawn could be said to fall within a certain interval.

A confidence level of 95 % has been selected for statistical analysis of experimental results. In most of the experiments conducted, ten specimens are used. For the others, groups of five or six specimens are used (there were limitations on the number of specimens that could be processed at a time for some of the experiments). Values of $t_{\alpha/2}$ are given in table 5.4.1 for different sample sizes (all at the 95 % confidence level).

n	$t_{\alpha/2}$
2	12.706
3	4.303
4	3.182
5	2.776
6	2.571
7	2.447
8	2.365
9	2.306
10	2.262
20	2.093

Table 5.4.1. $t_{\alpha/2}$ for different sample sizes at the 95 % confidence level.

It should be observed that the larger the sample size, the smaller the calculated confidence interval will be. This fact is reasonable, since the larger the sample size, the more closely that sample approximates the entire population.

This method of estimation is used to obtain a clearer idea of the behaviour of the polymer concrete specimens that have been investigated. It also proves to be a useful, quantitative way of expressing the reliability of the results obtained.

5.5 Characterisation of polymer concrete

5.5.1 Modulus of rupture and tensile strength

Using the 95 % confidence level, equation 5.1 predicts that there is 95 % probability that the mean load at failure in general of polymer concrete is between 1795 N and 1900 N. These values are calculated based on the typical flexural test results presented in table 4.1.1.

Using simple beam theory, the flexural stress required for failure of the beam specimens can be calculated: -

$$\sigma = \frac{My}{I} \qquad I = \frac{bd^3}{12} \text{ (for rectangular sections)} \qquad M = \frac{FL}{4} \qquad (5.2)$$

where M is the applied bending moment

F is the applied force in three point bending

L is the length of the span between supports in flexure rig

y is the distance from the neutral axis

I is the second moment of area of the specimen

b and d are the specimen width and thickness

For the case where F is critical (causes fracture), the applied moment M is taken as M_C (critical moment). The corresponding tensile stress on the underside surface of the specimen used is: -

$$\sigma_C = \frac{M_C(0.025)(12)}{(2)(0.025)^4} \qquad \text{and } M_C = F(0.035) \qquad (5.3)$$

then

$$\sigma_C = F(13440) \qquad (5.4)$$

This equation gives the modulus of rupture of the material in the configuration used. This figure is between 1.5 and 2 times the true *tensile* strength of the material (there is some uncertainty). The reason for this is that in a flexural test the whole section does not see the same stress, as is the case in a pure tensile test. The stress on the tensile surface of a beam in a flexural test is thus higher than the axial stress in a pure tension test. In addition it must be remembered that there may be a small amount of shear force acting. Substituting the mean failure load of 1850 N into equation 5.4, the modulus of rupture of this polymer concrete material is found to be 24.8 MPa. The 95 % confidence interval for this value is 24.1 – 25.5 MPa.

The tensile strength of the material then lies in the range of 12.4 – 16.5 MPa. Due to the uncertainty in the *tensile* strength values, modulus of rupture values (MOR) are given hereafter.

5.5.2 Fracture toughness

The mean value of K_{IC} for standard mix polymer concrete is found to be 1.85 $MNm^{-3/2}$, with a standard deviation of 0.27 $MNm^{-3/2}$ (see section 4.2.5). The 95 % confidence interval corresponding to these values is 1.65 – 2.00 $MNm^{-3/2}$. It can thus be stated that there is a 95 % probability that the stated interval contains the mean fracture toughness value of polymer concrete. Toughness is often expressed by the term G_C, which is related to K_{IC} and is given by: -

$$G_C = \frac{K_{1C}^{\;2}}{E(1+\nu)}$$

where E is Young's modulus, and ν is Poisson's ratio of the material. These two parameters were not determined during materials testing. Young's modulus is related to the elasticity of the material, and thus determines its rigidity or flexibility. Accurate determination of this parameter could not be done using the equipment available during the research.

Poisson's ratio for polymer concrete may be inferred from other tests involving E. The value of this parameter is always between 0 and 0.5. Brittle materials in general have low Poisson's ratios (0.2 – 0.25), while ductile materials have higher values (0.3 – 0.35). Poisson's ratio could not be directly measured for polymer concrete, but fortunately previous research on the subject is available and was consulted. The importance of Poisson's ratio to this investigation is that the stress analysis of manhole covers (using plate theory) involves equations that include ν. Equations giving the deflection of manhole covers involve E as well. Estimates of Poisson's ratio and Young's Modulus based on non destructive tests by Mantrala and Vipulanandan[4] are taken to be 0.22 and 18 GPa respectively.

Due to the uncertainties in E and v, G_C has not been calculated for polymer concrete. However, K_{IC} serves as a good indication of a material's resistance to crack propagation. The fracture toughness of polymer concrete is higher than that of cement concrete (0.2 – 0.5 $MNm^{-3/2}$), but lower than that of cast iron (6 – 10 $MNm^{-3/2}$). For this reason, polymer concrete is more susceptible to fracture by brittle crack propagation than cast iron. This point re-raises the issue of whether polymer concrete manhole covers can be successfully manufactured according to cast iron cover specifications.

5.6. Strength contributions of components in polymer concrete

Polymer concrete exhibits strain rate sensitivity, particularly at high strain rates. Required fracture loads are observed to increase when the load is applied at a faster rate. This behaviour is not likely to result in significant differences in performance of polymer concrete in manhole components, since the main mechanism of failure in this case is brittle fracture by cracking, rather than ductile yielding effects.

5.6.1 Chopped strand fibre reinforcement

Chopped fibre strands added to polymer concrete at about 0.5 % by weight of mix result in an increase in the modulus of rupture of the material from 20.8 to 24.8 MPa. The 95 % confidence interval for the mean modulus of rupture of non-fibre reinforced polymer concrete is 20.5 – 21.1 MPa. This reflects a strength increase of about 20 %. Chopped fibre additions also increase the fracture toughness (K_{IC}) of polymer concrete from 1.17 $MNm^{-3/2}$ to 1.85 $MNm^{-3/2}$ (approximately 50 %). The 95 % confidence interval for the mean fracture toughness value of polymer concrete without chopped fibre reinforcement is 1.14 – 1.19 $MNm^{-3/2}$. However, once the fibres are added to polymer concrete, they cause significant variance in the values from individual tests. This is due to the fact that they are randomly aligned in the material, and form 'clumps', particularly if mixed for too long.

The mechanism by which chopped fibres improve the mechanical properties of polymer concrete is that an applied load on the material as a whole is transferred to the glass fibres. Any deformation of the whole component requires that the fibres themselves deform, and due to their high strength and modulus, they resist this effect. Failure occurs when the applied load induces stresses in the material high enough to break the interfacial bond of the fibres to the matrix. In this case, fibres are pulled out of the matrix, and the strength of the material thus becomes dependent on the strength of the fibre/matrix interfacial bond. The fibres also resist the propagation of cracks through the material, since crack propagation requires local deformation of the material at the crack tip. The fibres resist this deformation, making it more difficult for a crack to propagate. This effect results in a general increase in toughness.

5.6.2 Reinforcement with woven roving matting

Woven roving reinforcement also results in a significant increase in strength of polymer concrete. Modulus of rupture values are observed to increase from 24.8 to 29.5 MPa. The 95 % confidence interval for woven roving reinforced polymer concrete for the tests conducted is found to be 27.5 – 31.5 MPa. The fracture toughness of reinforced polymer concrete is found to be 3.35 $MNm^{-3/2}$, with a 95 % confidence interval of 3.21 – 3.50 $MNm^{-3/2}$.

The observed strength increase is of the order of 20 %. It should be stressed here that this increase is exhibited by polymer concrete specimens of standard size and shape. It is doubtful whether this type of increase would be observed for polymer concrete manhole covers in which the arrangement of woven roving reinforcement (positioning, layering, etc.) is certainly not the same as that used for the specimens. It is certain however, that a significant increase in strength is achievable by incorporating this type of reinforcement into polymer concrete products. Table 5.6.1 gives a summary of the effects of various types of reinforcement in polymer concrete.

The strength improvements are dependent on the strength of the bond between the resin matrix and the fibres themselves. For this reason, good impregnation of the woven roving matting by the resin is required to optimise these strength gains. Accordingly, a higher resin content in the polymer concrete mix is required to ensure that proper impregnation of the matting does occur. The mechanism by which woven roving improves the strength and toughness of the material is that it increases the stress (and thus the load) required to *initiate* cracks. If cracks do appear, then the reinforcing effects of the woven roving are reduced. However, in order to improve the material it is required to improve its resistance to the initiation of cracks, since once they are initiated, their propagation is difficult to arrest.

	MOR (MPa)	95 % confidence interval (MPa)	K_{IC} ($MNm^{-3/2}$)	95 % confidence interval ($MNm^{-3/2}$)
Not reinforced	20.8	20.5 – 21.1	1.17	1.14 – 1.19
Reinforced with chopped fibres	24.8	24.1 – 25.5	1.85	1.65 – 2.00
Reinforced with woven roving and chopped fibres	29.5	27.5 – 31.5	3.35	3.21 – 3.50

Table 5.6.1. Summary of the measured mechanical properties of standard polymer concrete.

In general the addition of any type of fibre reinforcement to polymer concrete increases the resin requirements of that polymer concrete mix. It has already been found that these reinforcements do result in significant strength and toughness improvements and are thus vital to the performance of polymer concrete in load bearing applications. The problem with these reinforcements is that a higher resin content is required in the wet mix, and larger standard deviations in strength and toughness are observed in the hardened material. This in turn makes the mechanical properties of polymer concrete materials more difficult to predict.

Essentially then, fibre reinforced polymer concrete may be categorised as a composite material comprising a resin matrix, aggregate filler, and a reinforcement system. It exhibits rather *brittle* mechanical properties that are subject to significant variation. Unfortunately this variation seems to be largely unavoidable. However, certain degrees of confidence in mechanical properties can be expressed such that sensible design of polymer concrete products will result in reliable components that will not fail. Polymer concrete is certainly stronger in flexure than conventional concrete, but significantly weaker than cast iron. For this reason the *design* of polymer concrete manhole covers and frames becomes crucial to ensure reliability.

5.7 Viability of employment of different resins

In general, the results from section 4.3 imply that the orthophthalic resin UPE1623 gives perhaps the best overall performance in polymer concrete. The reason for experimenting with different resins is more to do with the fact that more than one resin supplier is desired to reduce delays caused by supply interruptions from any particular supplier.

The following resins give acceptable performance to polymer concrete.

- UPE1623
- Crystic 197
- Crystic waxed
- Polylite 33-410 (isophthalic)

These resins have been isolated from table 4.3.1 (section 4) as possible alternatives to use in polymer concrete. The DCPD resin gives reasonable modulus of rupture values but exhibits considerable brittleness. The other resins give MOR values that are too low for use in polymer concrete. The high standard deviation observed with the Crystic 197 resin is of concern, but as explained previously, this deviation probably has little to do with the resin itself, and should be attributed to the effects of the chopped fibres used in the polymer concrete mixes. Of the above mentioned resins, Polylite 33-410 gives the best overall performance in polymer concrete specimens, but is also the most expensive.

Other resin characteristics such as workability, gel time, ease of use etc. can reportedly be adjusted according to customer requirements, and as such are not of critical importance. Thus it appears that the resins listed above are suitable alternatives to UPE1623 that can be made use of in polymer concrete, without significantly affecting the mechanical properties of products.

5.8. Degradation by UV radiation

The results presented in section 4.4 confirm that polymer concrete is susceptible to degradation by UV radiation. It remains to establish the reliability of the results, and to equate them to effects in real time.

5.8.1 Strength loss due to UV radiation

The following table summarises the results obtained from both test 1 and test 2. Confidence intervals are not given here due to the fact that only six specimens could be exposed for each time. For the two week exposure time, the strength value given is the average of the two tests (1 and 2).

Exposure time (weeks)	Average failure load (N)	% strength loss
0	1840	(N/A)
1	(N/A)	(N/A)
2	1475	20
3	(N/A)	(N/A)
4	1600	13
5	1590	14
6	1570	14
7	1600	13
8	1390	24
9	1420	23

Table 5.8.1. Flexural strength loss due to UV radiation.

Figure 5.8.1 presents a graph of these combined values. It is suspected that the drop in strength after two weeks exposure to UV radiation was due to the positioning of those specimens in the UV exposure cupboard. On examination of the layout of the specimens in the cupboard, it was observed that specimens from Group 2 were unavoidably more directly exposed to the maximum power angle of the UV lamp. As a result these specimens received more radiation than the others over the same period.

A check on the positioning of specimens from other groups confirmed that they were more or less evenly spread out and should have received the same radiation levels.

The inconsistency results in the fact that Group 2 specimens from both Test 1 and 2 effectively received more radiation than they should have over the two week period. Nevertheless, it is of interest to note the substantial decrease in the modulus of rupture of the specimens after UV exposure.

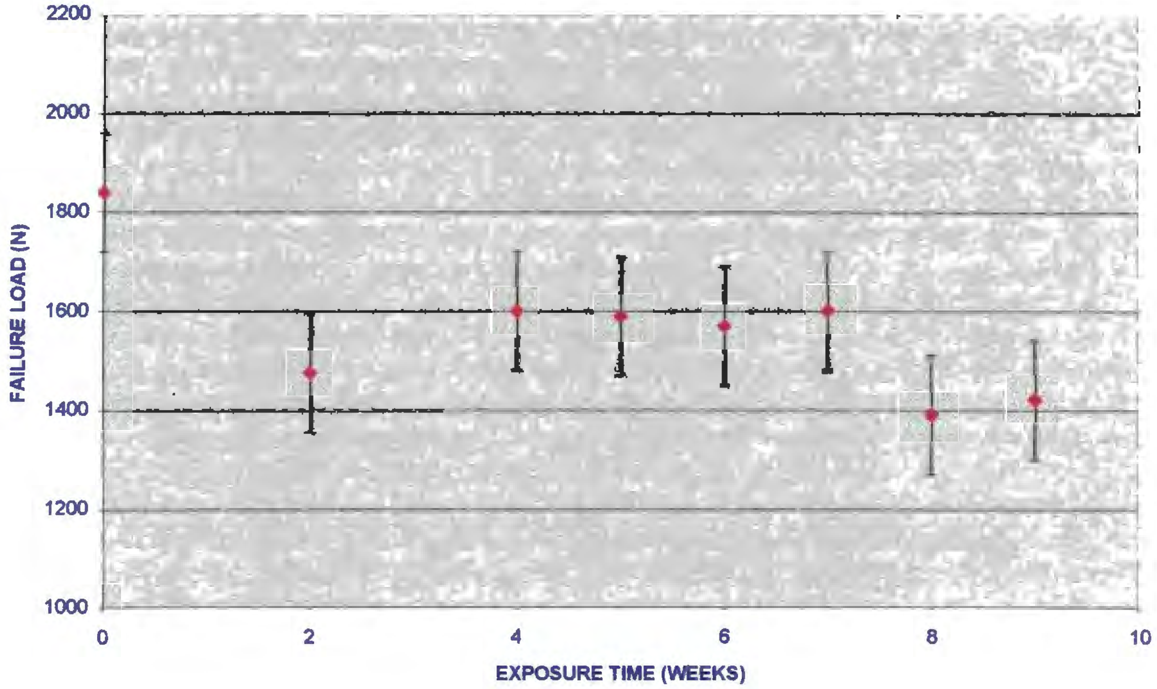


Figure 5.8.1. UV degradation of polyester polymer concrete.

5.8.2 Real time equivalence

The equating of these experimental exposures to real time exposure involves making use of a number of estimates. Exact figures cannot be determined here, but order of magnitude calculations are useful to present an idea of the nature of the problem.

The sun shines with a constant luminosity of 3.8×10^{26} watts at all wavelengths. Calculations show that over a sphere of radius 150 million km (distance to the sun), the radiation received by the earth is 1370 watts/m^2. This figure is known as the solar constant. UV radiation shorter than about 290 nm is absorbed by the ozone layer of the earth and therefore becomes negligible. The proportion of UV radiation between 290 and 400 nm that the sun emits may be approximated by integration of the energy – wavelength curve for the sun, which is similar to that of a black body radiator. At the temperature of the sun (5800 K), the percentage UV radiation of the *total* radiation emitted from the sun is approximately 12 %. Thus the UV radiation received by the earth is about 165 watts/m^2. The total power output of the UV lamp employed in the experiment was 125 watts, and this was spread over an area of 0.1 m^2. From the radiation information presented in figure 3.5.1, it appears that about 40 % of the radiation emitted from the lamp falls in the range of 300 – 400 nm. Thus the amount of UV radiation emitted by the lamp is found to be about 410 watts/m^2. This figure allows for the reflection of light inside the cupboard by foil paper on the inside walls. Thus the specimens inside the UV cupboard received about 2.5 times more radiation than they would have had they been exposed to the sun*.

The eight week exposure time then would have been equivalent to a continuous exposure time of 20 weeks (140 days) in the sun. The sun does not shine continuously however, nor are its rays directly impinging on the earth's surface at all times. Estimates can be made, but a small margin of error here (an hour or two) significantly affects the final time of exposure calculated. If it may be assumed that the sun shines (directly) an average of 8 hours per day, then the equivalent time of exposure in the sun is about 420 days (1.15 years).

The above analysis may be somewhat speculative due to the estimates made, however it does give a reasonable indication of the types of real exposure times being dealt with. It is thus estimated that the equivalent exposure time was about one year.

5.8.3 Significance of UV degradation

The following facts have so far been established: -

- UV degradation of polymer concrete *does* occur.
- Deterioration is time dependent.
- Strength losses of up to 24 % are possible.
- Real time exposure for a 24 % strength loss is about 1 year (for these specimens).

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* Quoted figures are based on data on the sun from Ref. 32 and assessment of the specifications supplied with the UV lamp used in the experiment.

It is likely that UV degradation of polymer concrete is a phenomenon that is largely limited to the surface of the material. For this reason, it is to be expected that the deterioration of mechanical properties of the material should also be limited to the surface. In the tests conducted, the specimen thickness was small compared to the thickness of components generally made with polymer concrete. Thus the loss in strength in polymer concrete manhole covers is likely to be limited to the top surface of the component rather than the regions subjected to high stresses.

The conclusion is that UV degradation of polymer concrete manhole covers is not as significant as the measured deterioration in the specimens tested, and failure of covers is not accelerated by UV radiation to any significant extent. Top surface embrittlement may result in spalling, chipping, and flaking effects after long exposure times. These phenomena may occur due to high surface compressive stress generated during loading, and will be accelerated by UV embrittlement effects.

5.9 Degradation by solvents, acids, and alkalis

The results presented in sections 4.5 and 4.6 confirm that degradation by certain chemical environments can occur. In particular, considerable strength losses have been observed for short exposures to sodium hydroxide solution, and to a somewhat lesser extent for sulphuric and hydrochloric acids over the same exposure periods. Long exposures to water and saline solution also produce significant falls in flexural strength of polymer concrete.

5.9.1 Strength deterioration by chemical environments

Table 5.9.1 presents a summary of the results of exposure to chemical environments after relevant exposure times. Only the longest exposure times for the tests are listed (except for water and saline solution). The results indicate that alkaline environments present the most serious threat to polyester polymer concrete. This is consistent with previous research that has found that polyesters are susceptible to attack by alkalis (and acids to a lesser degree). In polymer concrete, the aggressive environments tend to attack the resin matrix/aggregate interface rather than the resin itself. Porosity in the material thus accelerates the degradation process substantially. Water is known to degrade polyester polymer concrete, but not polyester resin on its own. This fact serves to confirm that it is the interfacial surfaces that are attacked rather than the material as a whole.

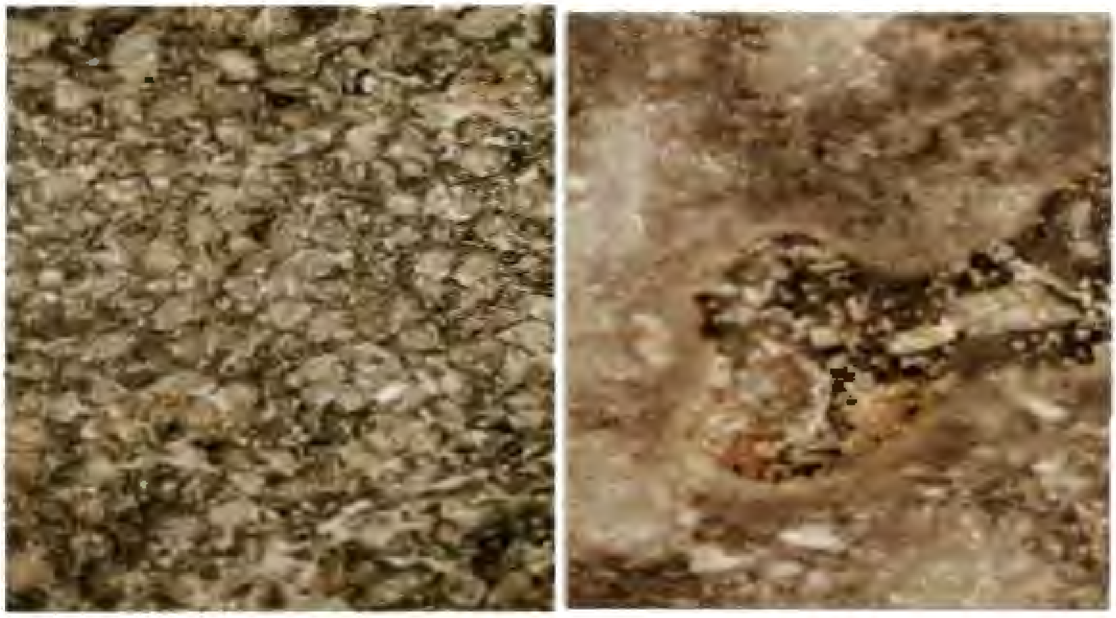
Environment	Description	Exposure time	Average failure load (N)	% strength loss
Gasoline	volatile solvent	3 weeks	1850	0
Diesel fuel	volatile solvent	3 weeks	1800	3
Engine oil	synthetic oil	3 weeks	1850	0
Water	solvent	3 weeks	1800	3
Water	solvent	7 months	1460	21
Saline solution		3 weeks	1500	18
Saline solution		7 months	1200	35
Sulphuric acid	acid, pH 1.5	15 days	1500	18
Hydrochloric acid	acid, pH 1.5	15 days	1400	24
Sodium hydroxide	alkali, pH 13	15 days	1150	38

Table 5.9.1. Degradation by various chemical environments.

In addition, no measurable weight loss in the specimens is recorded for the majority of experiments. In the case of sodium hydroxide, a 0.5 % weight loss is recorded. Lightening in colour of the specimens can be attributed to the effect of interface attack, ie. the aggregates and fibres become more visible through the gelcoat.

5.9.2 Significance of strength deterioration

Degradation by these environments poses a more serious threat to the integrity of polymer concrete manhole covers than does UV degradation. The reason for this is that aggressive environments frequently come into contact with sections of a component already under stress. Degradation of these parts of a manhole cover can certainly result in higher susceptibility to crack propagation in these critical regions. Observation of various covers in service revealed that waste products flowing underneath a manhole may condense on the underside of the cover, and in particular, in the corner of the lip section. Attack in these regions accelerates the initiation and formation of cracks, and ultimately leads to premature failure of the cover. The process in some ways is analogous to stress – corrosion cracking (SCC) in metallic materials. The idea that it is the aggregate – resin interface that is attacked is illustrated in figure 5.9.1 (a). The photograph is of a specimen that had been immersed in saline solution for a period of seven months. It appears that some sort of deposit has formed around the aggregate particles that may account for the observed strength loss. The photograph in figure 5.9.1 (b) is a closer view inside a cavity on the surface of the same specimen. Tiny yellowish deposits are visible on the surface of the aggregate particles. As yet their composition has not been conclusively determined, but it is likely that they may be salt crystals that have formed between the aggregate and the resin surfaces. The formation of these crystals introduces stresses between the two surfaces, weakening the interfacial bond.



(a) *(b)*

Figure 5.9.1 (a) and (b). Deposits on aggregate particles after exposure to saline solution for seven months. (a) at 9X magnification, (b) at 55X magnification.

Exposure to saline solution reveals that polymer concrete is not suitable for use where exposure to salt solutions is required. In addition polymer concrete should not be used for purposes involving partial, or complete submersion in water. As with UV ray effects, it is likely that degradation by solvents and aggressive environments is limited to the surface regions of the material, although due to porosity, the effect may extend slightly further into the material. The critical point is that these aggressive environments come into contact with areas of manhole covers already under high stress, whereas UV rays only affect the top surface of covers in service.

5.10 Polymer concrete with graded aggregates

There are two main reasons for experimenting with graded aggregate systems: -

- To reduce the void content of polymer concrete products (to improve strength).
- To reduce the resin content of the mix (to save cost).

The results presented in table 4.6.1 in section 4.6 show (in general) that the use of graded aggregates in conjunction with fibre reinforcement in polymer concrete is not viable at this stage. The main reasons for this are as follows: -

- Large aggregates are considerably more expensive than finer aggregates.
- No improvement in strength is observed in most cases. In some systems a substantial loss of strength is observed.
- The use of graded aggregates considerably complicates the manufacture of polymer concrete.
- No significant reduction in resin content could be achieved.

Taking all factors into account, it seems that at present the use of graded aggregate mixtures is more detrimental to polymer concrete than beneficial. It is thought that the chopped strand fibre additions are the main reason why a reduction in resin content cannot be achieved. It has been shown throughout the course of this research that chopped strand fibre additions increase the required resin content of the mix.

The large particles used in these experiments are all angular in nature (rounded particles of the required size were not available). This is significant and it is thought that the shape of these particles tend to result in stress concentrations at sharp corners, substantially weakening the material.

The graph in figure 5.10.1 gives an idea of the effect on flexural strength of increasing proportions of angular shaped coarse aggregate in fibre reinforced polymer concrete mix. For this analysis, No.1 Grade and No.2 HP Sand are classed as fine aggregate, while 4/10, 7/16, and 5-6.7 mm Filter grits are classed as coarse aggregate.

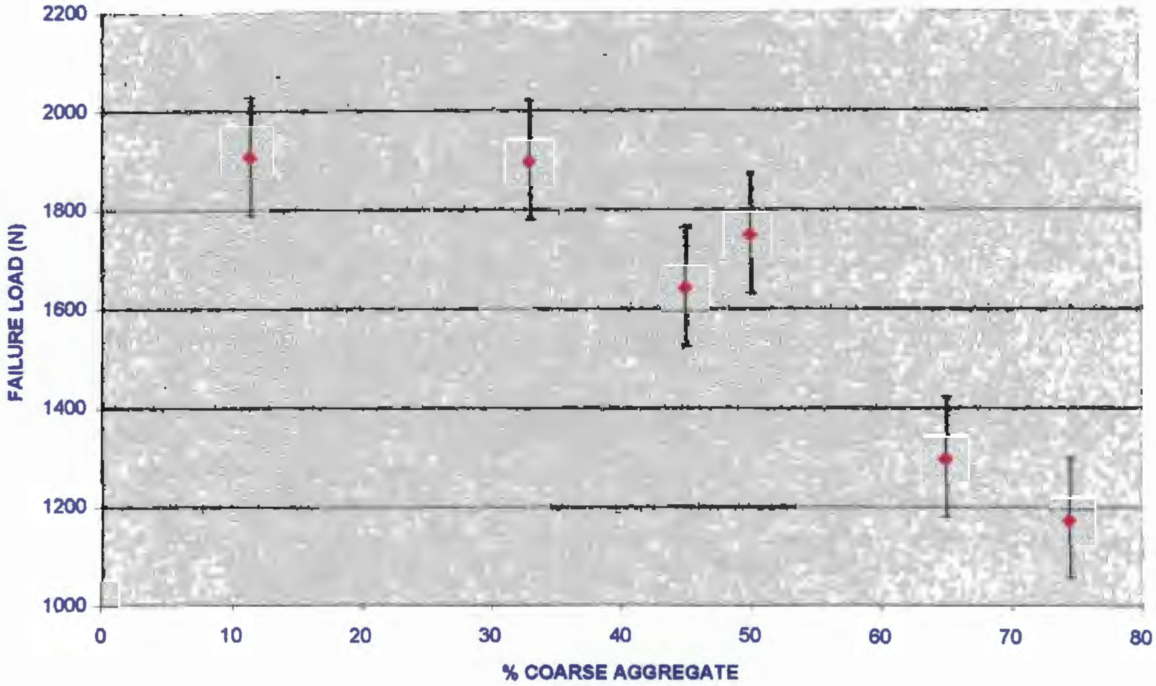


Figure 5.10.1. Effect of increasing percentages of coarse angular particles in aggregate graded polyester polymer concrete specimens

For the reasons already discussed, the use of a graded aggregate system (of angular particles) in conjunction with chopped strand fibres in polyester polymer concrete is not viable at this stage.

5.11 Viability of silane coupling agents

Silane additions to polymer concrete result in significant improvements in the flexural strength of the material. It is thought that silane coupling agents improve the bond between the resin and the aggregate, and the bond between the resin and the glass fibres. These compounds have a molecular structure that facilitates a 'link' between the different components of polymer concrete. Any improvement in interfacial adhesion increases the strength of the material. Investigation and experimental procedures have revealed the following: -

- The addition of 1 – 2 % silane in general improves the flexural strength of polymer concrete by approximately 16 %.

- Better strength gains are obtainable by ***pre-treating*** the resin or the aggregates with the silane prior to mixing, rather than by adding the silane directly to the mix.
- The usual large standard deviations are observed.
- The addition of 1 % silane (by weight of resin) to polymer concrete mix improves the fracture toughness of the material slightly, from 1.85 to 1.98 $MNm^{-3/2}$.
- The optimum amount of silane that should be added to polymer concrete is about 1%. No significant strength improvements are observed in specimens containing more than this amount.
- The silane coupling agent used in this investigation has a product code GF31 and is expensive. Price quotes of around R250-R270/kg have been obtained. As yet it is not known whether prices are negotiable.

The strengthening effect of silane coupling agents is significant. It is certain that products manufactured from polymer concrete with these additions will be more reliable than products manufactured from conventional polymer concrete. They will be stronger in flexure and more resistant to failure by crack propagation. The only disadvantage of silane coupling agents is their cost, and it seems this factor renders the use of silanes in polymer concrete manhole covers not viable at this stage.

One other important aspect of the silane investigation that has been noticed is that the mixes incorporating silane seem to be 'wetter' than normal. This led to further thoughts of possibly reducing the resin content of the mix, saving this expense, and thus making the extra cost of the silane viable. However, it has been established that the saving obtained by reducing the resin content in a silane treated mix is small, and does not offset the extra cost of the silane.

5.12. Viability of postcuring

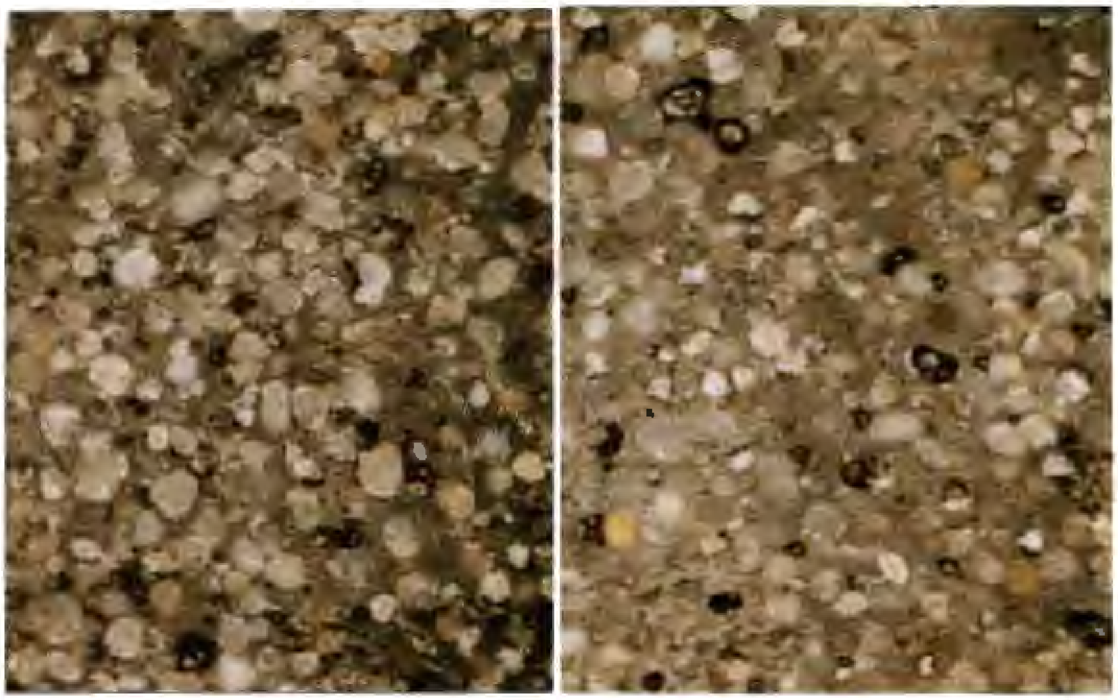
The results presented in section 4.8 have established that the postcuring methods recommended by the resin suppliers have little or no beneficial effect on the flexural strength when applied to polymer concrete. It seems that the main reason that postcuring procedures are recommended by resin manufacturers is that the rigidity of the cured resin is improved. Postcuring affects the *state of cure* of the resin, which in turn affects the rigidity or modulus of the resin, rather than it's flexural strength. Indeed, higher rigidity of the postcured polymer concrete specimens is observed during testing. However, this higher rigidity gives no improved resistance to the nucleation and propagation of cracks. Rather, it is thought that the higher rigidity may render the material *more* susceptible to cracking and brittle fracture (under the loading conditions a manhole cover is subjected to).

It is also taken into account that the incorporation of postcuring procedures into the manufacturing process at the AV Mouldings factory would significantly increase the cost of production of each polymer concrete product. However, once installed, the process would not necessarily lengthen the production cycle. Based on the results of postcuring tests presented in section 4.8, it seems that postcuring methods are not beneficial to products, and therefore further investigation of this process is not considered to be worthwhile.

5.13 Viability of employing vibration techniques

It has been found from the results of section 4.9 that vibration methods appear to be detrimental to the flexural strength of the polymer concrete specimens tested. This goes against the theory that void removal by vibration methods should improve the mechanical properties of polymer concrete. It is believed that the problem with these methods is that they also result in segregation of the aggregates and the resin. This theory is confirmed on examination of figures 5.13.1 (a) and (b).

In these photographs it is immediately evident that there is a higher aggregate density in the bottom half of the specimen than in the top half. In addition it appears that numerous voids are still present in both pictures. Essentially, the vibration method is not successful for the specimens used. In thin moulds, there is restriction on the diffusion of air bubbles to the surface, while wider moulds allow more air bubbles to reach the surface. The fact that there is a significant *drop* in strength due to vibration methods is directly due to the segregation issue already mentioned.



(a) *(b)*

Figure 5.13.1. Cross section of vibrated specimen. (a) Bottom half, (b) top half.

The bottom half of the specimen has become depleted in resin, while the top half has become resin rich. In these specimens, the main tensile stresses are induced on the bottom surface by the three point bend test configuration, and it is exactly these regions of the specimens that have become depleted in resinous binder. Consequently the interfacial bond strengths are not as high and a lower failure load is then measured.

It is believed that the segregation effect may result in these vibration methods not being viable to employ. Even in larger mouldings where there is less resistance to air bubble diffusion through the mix, it is to be expected that segregation will occur to the same extent, if not more so.

A possible solution to this problem may be in the use of thixotropic resins, which would restrict the segregation effect. However, it is also likely that air bubble diffusion in the mix would be similarly restricted, and little improvement could then be expected. The use of these resins would also complicate the rest of the manufacturing process, reducing workability and thus the mouldability of the mix.

6. LOADS AND STRESSES ON MANHOLE COVERS

This section presents a detailed analysis of the loads and stresses that a manhole cover may be subjected to in service. The emphasis is on the Type 2A replacement cover that AV Mouldings currently manufacture from polyester polymer concrete. As established previously, it is mainly these components that have been known to undergo premature failure in service.

6.1 Loading conditions

The loads that are applied to manhole covers found in roads are subject to variation in magnitude, rate of application, time of application and rate of repetition. As always it seems sensible to consider worst case scenarios when attempting to determine the performance of products in service. In addition, the use of estimates becomes unavoidable when dealing with problems of this nature. However, any estimates made in this analysis are thought to err considerably on the side of safety.

The maximum conceivable mass that a heavy vehicle (articulated trucks and pantechnicons) may have is estimated to be 50 metric tons. It is possible that certain vehicles may have masses even higher than this value, but these conditions are considered to be highly abnormal. Due to the configurations of some articulated trucks, the total mass may be divided between trailers, and the wheel loads are divided through the number of wheels per trailer. For this analysis, the full weight of a heavy 14 wheeled truck is divided by the number of wheels to determine the maximum load that a manhole cover may be subjected to. The weight per wheel is then found to be about 40 kN. Conceivably, for some double wheeled trucks, the actual load that may be applied to a manhole cover can be about 80 kN. This figure is considered to be a good estimate of the maximum loading condition possible, although it is likely to be significantly larger than the true value for most loading conditions.

The time of loading and rate of application of this load is dependent on the size and speed of the vehicle, as well as the diameter of the manhole cover itself.

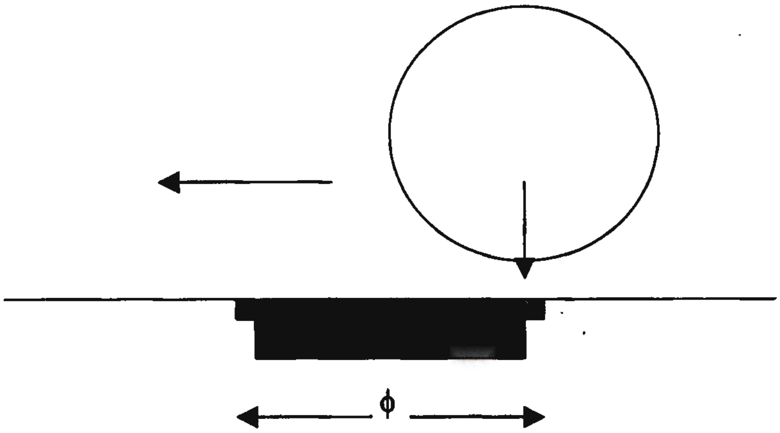


Figure 6.1.1. The nature of loading on a Type 2A manhole cover

In general, the wheel shape is flattened on its bottom surface, resulting in the applied load being spread out over an area over the cover. Figure 6.1.1 illustrates the simplified loading condition on the cover. The time required for the wheel to completely pass over the cover is given by: -

$$T = \frac{\phi}{V}$$

where ϕ is the diameter of the cover and V is the velocity of the truck. Using the diameter of a Type 2A cover (0.630 m) and a truck velocity of 120 km/h (33 m/s), the total period of loading is found to be 0.02 seconds (per wheel). The maximum load on the cover will occur at time T = 0.01 seconds. This approximation can be made since the area of application of the load in practice is larger than the area covered by half the diameter of the cover. For the 50 ton truck previously discussed, moving with constant speed, V, the rate of application of this load may be more or less linear and is found to be 8000 kN/sec. Various different truck speeds may affect this value, however it is considered to be a good estimate of the worst case loading scenario.

The rate of repetition of loads is certainly dependent on the type of traffic flow for a particular road. In addition not all vehicles travelling along a road will necessarily pass over a manhole cover in that road. For this reason a certain amount of speculation may be involved, but it is still interesting to make use of estimation methods to get an idea of the nature of the problem. If it can be estimated that for a road subject to heavy traffic flow during the day and lighter flow at night, that the average rate of vehicles passing over a cover is 1 vehicle per minute, we arrive at a figure of about 530 000 vehicles per year. The number of actual cycles of loading that this figure gives is dependent on a value equal to ***half*** of the average number of wheels per vehicle. Four wheeled cars are far more common than 22 wheeled trucks, thus an average value of 6 is selected. The number of loading cycles per year is then estimated to be 1.6 million. It must be remembered that few of these loading cycles will be of high magnitude. Similar analysis may be performed for the many types of traffic flow that are possible.

6.2. Stresses on Type 2A polymer concrete manhole covers

The loads applied to manhole covers induce stresses within the material that may be found using circular plate theory (section 2.8). This theory predicts that the maximum stress on a circular plate, of constant thickness simply supported around its circumference and subjected to a distributed load, will be at the centre of the plate on its surface. The nature of the stress will be tension on the underside and compression on the upper side. The stress and deflection distribution may be calculated using equation 2.1 varying r from 0 to a (full radius of the cover).

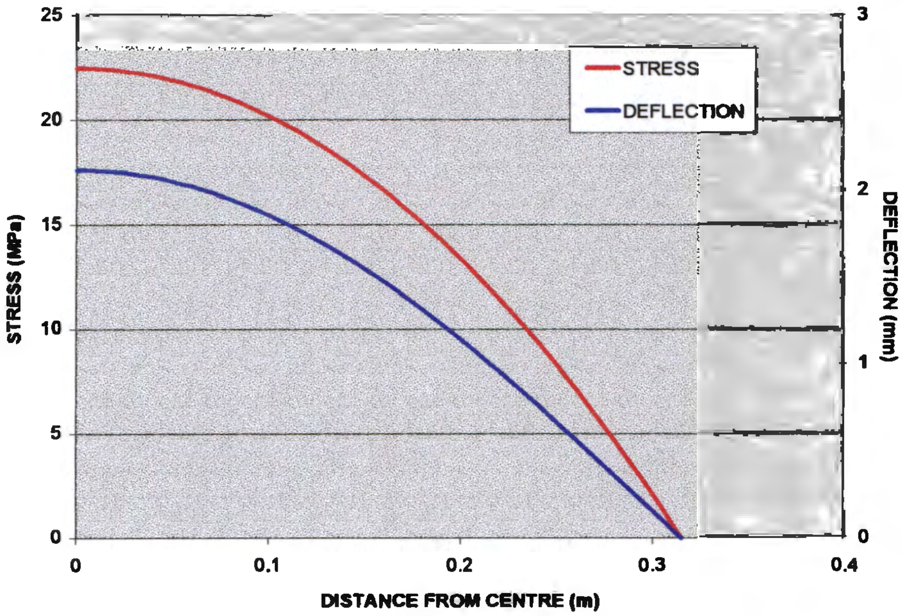


Figure 6.2.1. Stress and deflection distribution for a simply supported circular plate of radius 315 mm and thickness 37 mm, with a uniformly distributed load of 80 kN.

In figure 6.2.1, the thickness of the plate is taken as that of the lip section only ($t = 0.037$ m). E and ν are taken as 18 GPa and 0.22 respectively, and the applied load is taken as that of the worst case scenario (80 kN) spread over the total area of the cover. It is found that the value taken for ν does not significantly affect the overall stress distribution. The value taken for E does not affect the stress distribution, but has a noticeable effect on the calculated deflection curve of the plate.

The Type 2A manhole cover consists essentially of two connected circular plates. It may also be considered to be a single circular plate of non-uniform thickness. The discontinuity in thickness is found in the lip region of the cover and results in significantly higher stresses there than would normally be expected. The change in thickness complicates the derivation of the stress condition on the plate, however due to the fact that the difference in radius of the bulk section compared to the lip section is small (less than 8 %) an *approximate* stress distribution for the cover may be obtained. This approximation is a good estimate of the worst case scenario. Figure 6.2.2 illustrates the discontinuity in stress level that is seen in the lip region due to the sudden change in plate thickness. The stress in this region is certainly higher than the stress at the centre of the plate.

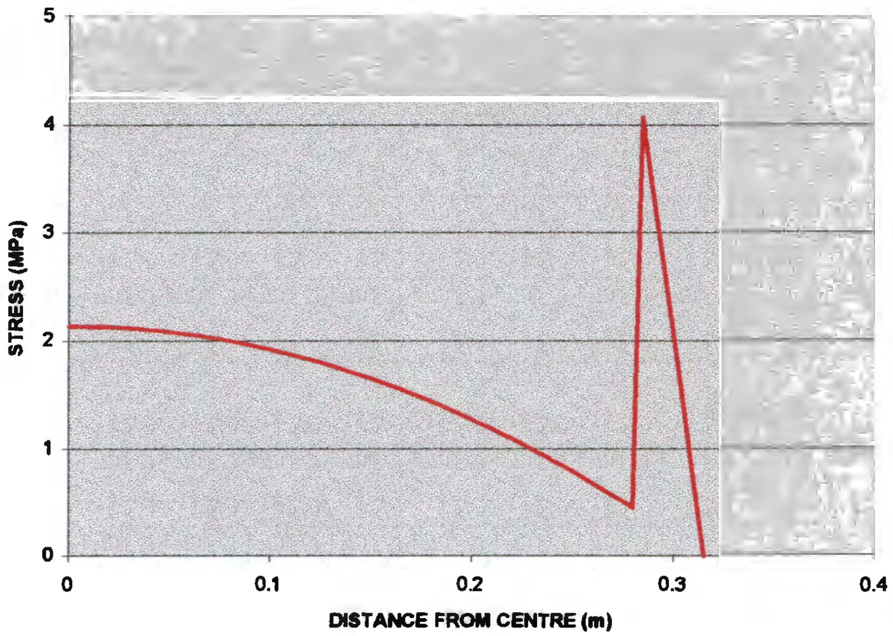


Figure 6.2.2. Approximate stress distribution in the modified Type 2A replacement manhole cover with a uniformly distributed load of 80 kN.

In figure 6.2.2, the thickness of the cover is taken as that of the whole section (t = 0.120 m) up to the radius at which the section thickness becomes that of the lip section only (0.037 m). All other parameters remain the same as those taken in figure 6.2.1. The effect of the change in section thickness at the bulk/lip intersection is to produce stress concentration in this area. The small radius often employed in the lip region of these covers further increases the stress concentration.

The strength properties of polymer concrete are summarised in section 5.5, and it appears that the tensile, compressive, and flexural strengths of polymer concrete are above the **macro** stress levels generated due to heavy vehicle traffic flow. This is consistent with the fact that in general most manhole covers are considerably overdesigned. The fact remains that some polymer concrete manhole covers have failed by fracture processes at stresses well below the static strength of these components. It is believed that the small radius of curvature between the lip and bulk sections induces **local** stresses, which may be higher than the static strength of the material. Repetitive heavy loading cycles then cause cracks to be initiated in this area followed by crack propagation, which ultimately leads to catastrophic failure of the entire component. It has also been suggested that in some cases the Type 2A replacement covers do not seat properly in the cast iron frames, resulting in the cover rocking during loading. The effect allows the cover to undergo larger deflections than is usually the case, and consequently the initiation of cracks is accelerated.

6.3 Solutions

AV Mouldings have already begun manufacture of their own design of a Type 2A manhole cover and frame. The difference in these designs is that the lip section is thicker while the bulk section is thinner. The product complies with minimum weight requirements, and as yet has suffered no failures in service. These covers cannot be used to replace stolen Type 2A cast iron covers unless the entire existing cast iron frame is removed from the road and replaced with the appropriate polymer concrete frame. This procedure is generally costly and has so far been avoided where possible.

The photographs below illustrate the two different shapes for the Type 2A manhole cover. The increased thickness of the lip section in the AV Mouldings designed Type 2A cover is clearly seen.

Figure 6.3.1. The standard Type 2A 'replacement' manhole cover (a modified version of the SABS Type 2A cast iron cover).

Figure 6.3.2. The AV Mouldings designed Type 2A manhole cover.

6.3.1 Stress analysis of the AV Moudings designed Type 2A cover and frame

The new design currently being used by AV Mouldings in cases where both a cover and frame are to be installed is shown schematically in figure 6.3.3. In this case the thickness of the lip section is great enough to ensure that failure does not occur. The lip section is far more rigid, reducing the 'peeling' tendency described in section 5.2.1. The stress curve for this design is calculated in the same way as that for the modified Type 2A replacement cover, and is plotted in figure 6.3.4.

Figure 6.3.3. The shape of the AV Mouldings designed Type 2A cover.

This design is based on the original Type 2A cover design (see Appendix C), ie. basic dimensions are similar, only the thickness of the lip section has been substantially increased. As already mentioned, these covers have performed well in service, and are recommended in cases where both cover and frame are to be installed.

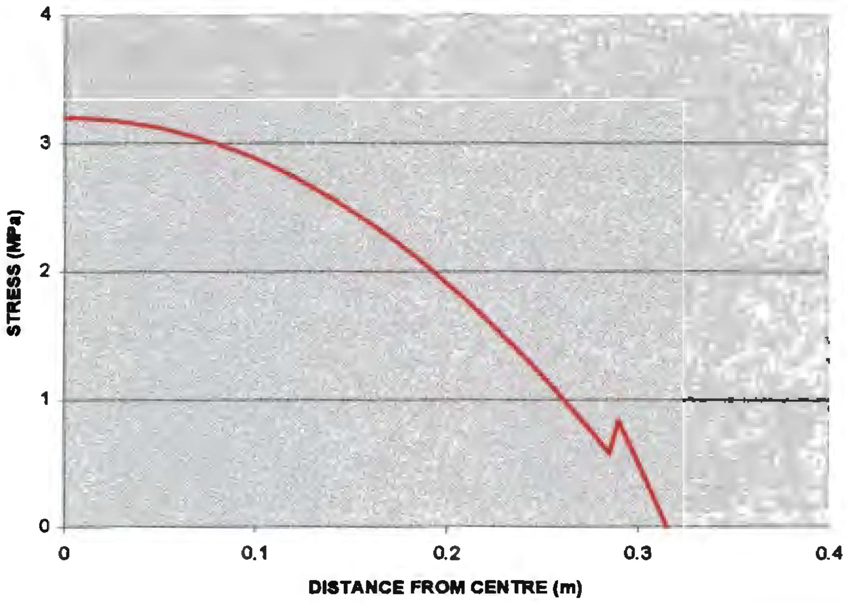


Figure 6.3.4. Approximate stress distribution in the AV Mouldings Type 2A manhole cover, with uniformly distributed maximum load of 80 kN.

It should immediately be observed from figure 6.3.4 that the stress concentration at the intersection of the lip and bulk sections has been dramatically reduced. The reason for this is the fact that the stress in a circular plate varies with the inverse of the thickness squared. By employing a thicker lip section it is possible to reduce the stresses in this region to values similar to those at the bulk section interface. The chances of failure of a cover of this design in the lip region have become negligible.

The region of highest stress in this cover design is in its centre, and it may be noticed that this stress level is actually higher than that in the centre of the modified Type 2A replacement cover. However this higher stress level is still much less than the fracture strength of the material. Failure through the centre of the component is also unlikely with this design. The important point is that the stress concentration in the lip region has been removed.

6.3.2. Possible solutions to failures of standard Type 2A replacement covers

A number of possible solutions have been envisaged to deal with the failure of standard Type 2A covers (intended to be *replacements* for stolen cast iron covers). It has been established that the small radius under the lip section is responsible for stress concentrations that increase the local stress in this region. The following designs have so far been considered to reduce these stress concentrations.

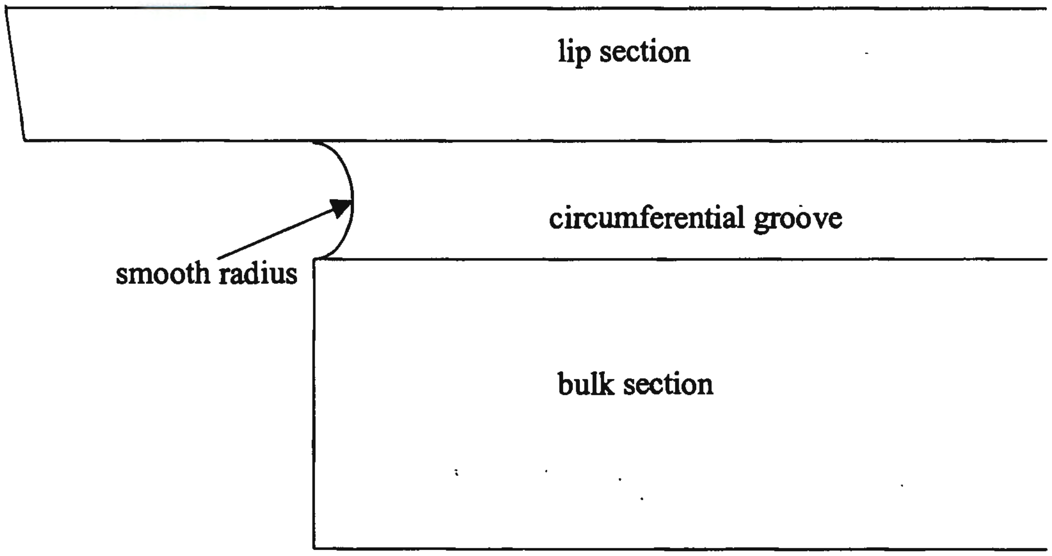


Figure 6.3.5. The ring groove design conceived to reduce stress concentrations.

In the current Type 2A cover design, the radius under the lip cannot be simply enlarged, since its fit into the existing frame must be taken into account. The introduction of a radius onto the support section of the cast iron frame to facilitate the fitting of a redesigned Type 2A cover has been considered. However, it has been established that this machining process would cost more than the process of lifting up the entire cast iron frame, and replacing it with the new AV Mouldings design. The design presented in figure 6.3.5 is simple to manufacture, and requires slight modifications to the existing moulds (to facilitate removal of products from the moulds). The size of the radius shown can be varied using different moulding techniques to analyse the response of the component to repetitive loading. It has been established during preliminary calculations that the radius should be at least 15 mm.

Various other designs have been considered. They are presented in figure 6.3.6 and figure 6.3.7.

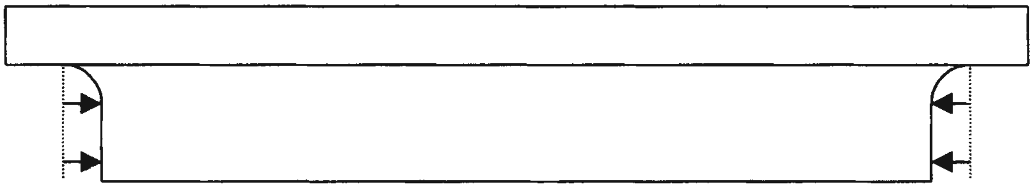


Figure 6.3.6. Reduction in bulk section radius to facilitate the fitting of a smooth radius corner under the lip section.

The problem with the design shown above is that the side walls of the bulk section are not firmly located by the walls of the frame. The lip section sides are located around the circumference as usual, but it is suspected that a cover like this that is not properly located throughout it's thickness would be likely to bounce out of its frame.

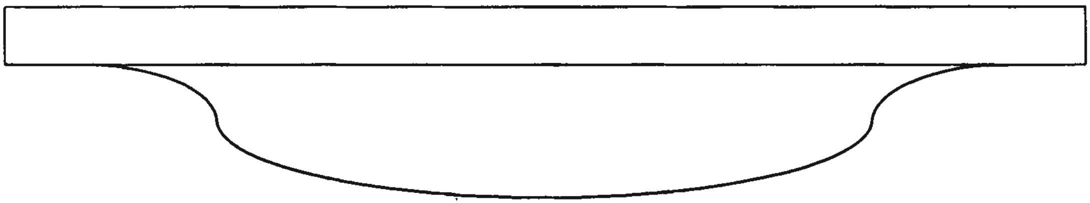


Figure 6.3.7. Hemispherical bulk section with smooth radius under lip section.

Again there is a location problem with the design in figure 6.3.7. A component of this design is also considerably more difficult to manufacture and requires complete redesign of the current moulds as well.

Further methods to avoid the initiation of cracks underneath the lip section include the use of a galvanised steel 'mould' plate that fits around the circumference of the component. The polymer concrete mix would be moulded into this steel mould which would become part of the moulding. The idea is illustrated in figure 6.3.8. The benefits possible by making use of this type of reinforcement result from the fact that the steel plating has a much higher fracture toughness than the polymer concrete, and thus requires larger stresses in the lip region to initiate cracking.

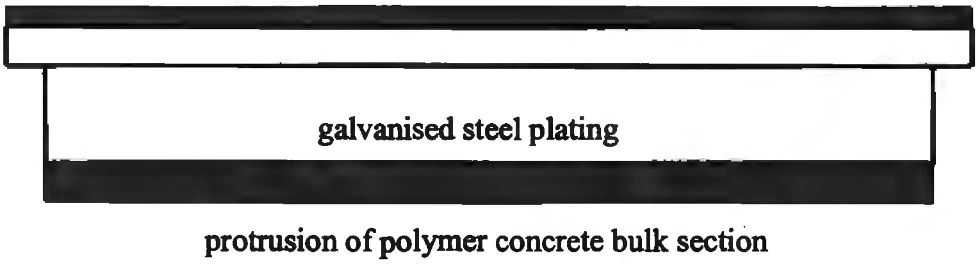


Figure 6.3.8. Polymer concrete manhole moulding with galvanised steel surface reinforcement.

The regions of highest stress here are still under the lip region, but the steel plating acts as a protector of the polymer concrete bulk moulding. The problem with this configuration is that in order for it to work, absolute adhesion of the polymer concrete bulk material to the galvanised steel plating is required. If this is not the case, and the two surfaces separate, then the loading on the top surface of the cover is transferred directly to the lip region, and the reinforcing effect of the steel plating is lessened. It is likely that should the two materials become separated at any point, severance will ultimately propagate completely around the circumference of the component. In this condition, the steel plating becomes useless, and provides very little resistance to crack initiation and propagation in the polymer concrete itself.

The important issue is then to achieve good adhesion of the polymer concrete to the galvanised steel plating. Various methods such as surface roughening, and even certain types of coupling agents have been considered, however the two materials are inherently not compatible with each other. For this reason, the above method of reinforcement, while extending the life of a component slightly, does not appear to be a viable solution at this stage.

Various other reinforcing methods have been proposed. These include the use of steel mesh, steel rods, and other forms of metallic 'fibres'. At present the viability of these forms of reinforcement has not been established, but preliminary considerations show that it is doubtful how much benefit they may have. The basis for this reasoning is the fact that good adhesion of the polymer concrete material to the steel reinforcement cannot be ensured, and without proper adhesion the reinforcements become useless.

The most viable solution appears to be the design presented in figure 6.3.5. It is believed that a component with a radius under the lip of at least 15 mm (but at most 20 mm) would perform better in service than the current design. At present AV Mouldings are experimenting with methods of employing this design for the Type 2A replacement cover.

7. CONCLUSIONS AND RECOMMENDATIONS

The results of the research have led to the belief that although polymer concrete may be suitable for use as a material in load carrying applications, simple design methods should be employed to ensure the reliability of such components. This section is a brief summary of the more important points discovered during the course of the work. These points pertain to polymer concrete generally defined by table 2.5.1.

1) Polymer concrete can be classed as a composite material that exhibits brittle characteristics. Failure occurs by brittle crack propagation.
2) The use of glass fibre reinforcement increases the MOR of the material from about 21 MPa to about 25 MPa. Adding woven roving reinforcement to polymer concrete beam specimens further increases the MOR to about 30 MPa.
3) The fracture toughness of unreinforced polymer concrete is found to be 1.17 $MNm^{-3/2}$. This is increased to 1.85 $MNm^{-3/2}$ by adding chopped fibres, and then further increased to 3.35 $MNm^{-3/2}$ by employing woven roving in beam tests.
4) The use of glass fibre reinforcement in polymer concrete while improving the strength and toughness of the material, also affects the workability of the mix, limiting the minimum resin content to about 19 %. This figure varies for other mixes according to the volume fraction of the fibres in the mix.
5) The addition of glass fibres to polymer concrete introduces significant statistical variation to the (improved) strength.
6) The addition of 1 % silane coupling agent to fibre reinforced polymer concrete increases the MOR of the material from 25 MPa to about 29 MPa. The fracture toughness is increased from 1.85 $MNm^{-3/2}$ to 1.98 $MNm^{-3/2}$.
7) In general, the use of graded aggregate mixtures is detrimental to the mechanical properties of glass fibre reinforced polymer concrete. It is concluded that this is due to the angular nature of the larger particles employed. Sharp corners and angularities cause internal stress concentrations that reduce the effective strength of the material.
8) Polyester polymer concrete is susceptible to degradation by acids, alkalis, water, and salt water. The mechanism is thought to involve the diffusion of the aggressive solution through the material followed by attack of the aggregate –

resin and fibre – resin interfaces. The interfaces are particularly susceptible since these areas are surfaces with a higher energy associated with them than the surrounding matrix, and liquid solutions will tend to collect there due to surface tension, particularly if there is poor interfacial adhesion. This degradation is a cause for concern, and does affect the reliability of polymer concrete components.

9) UV radiation has been found to degrade polyester polymer concrete specimens markedly. However, it is concluded that any degradation is confined to the surface of the material, and is thus not considered to be a critical problem.
10) Postcuring of fibre reinforced polyester polymer concrete does not appear to improve the flexural strength of the material.
11) At present, the use of vibratory moulding methods seems to be detrimental to the flexural strength of fibre reinforced polymer concrete specimens. It is believed that this is due to segregation of the aggregate and the resin during vibration. In addition, the diffusion of air bubbles through the viscous mix in thin specimens (such as the ones used) appears to be hampered by the restriction of the mould walls.
12) The failure of Type 2A replacement manhole covers is due to the abrupt change of thickness and small radius of the lip section. The effect of the high strain rate and repetitive nature of the loading condition is thought to further increase local stresses, which can approach the calculated brittle fracture strength of the material during high loading. Improper impregnation of woven roving reinforcement can create an internal surface through which crack propagation is facilitated.
13) Once cracks have been initiated in polymer concrete, the stress required for them to propagate is vastly reduced. For this reason it is desired to reduce the possibility of crack initiation altogether.

A combination of degradation factors together with the stress concentration associated with design flaws in the lip section are the direct cause of premature failure of Type 2A replacement manhole covers made from polymer concrete. This does not mean that polymer concrete is not suited for use in manhole covers, since the AV Mouldings Type 2A manhole cover and frame have performed well in service. If continued production of the Type 2A replacement cover is desired, then a certain amount of redesign of the cover based on the results of this report is required.

Since no SABS specification exists for these components, the following points should be remembered during the manufacture and use of polymer concrete components. Full recommendations for an SABS specification are given in Appendix B.

1) The current polymer concrete mix produced by AV Mouldings *is* conditionally suitable for use in manhole covers and frames.
2) Fibre reinforcement is crucial to the flexural strength and toughness of polymer concrete. Under no circumstances should unreinforced polymer concrete be used for Type 2A manhole covers (or frames).
3) Postcuring procedures, vibratory moulding methods, and graded aggregate mixtures (of angular particles) should not be employed at this stage.
4) The use of silane coupling agents in fibre reinforced polymer concrete is recommended. However, cost factors may render these products non-viable.
5) Redesign of the Type 2A replacement cover is required to ensure reliability. A design based on the schematic in figure 6.3.5, employing a radius of at least 15 mm (but not more than 20 mm) is recommended.
6) Current Type 2A replacement covers should not be used in service where the pH level of the effluent moving through the drainage system exceeds 9. Failure of components under such conditions is inevitable even at lower loading.
7) Redesigned Type 2A replacement covers in service should be inspected at three month intervals for signs of crack initiation.

It is believed that polymer concrete can be successfully used to manufacture manhole and drain components. It remains to define testing procedures and quality control methods to ensure that these products comply with SABS specifications that are to be drawn up. Recommendations for this specification are given in Appendix B.

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APPENDIX A: Stress calculations of manhole covers in service compared to current SABS testing procedure.

Type 2A manhole cover in service

Conditions: - Covers are considered to be simply supported around the edges
subjected to a rapidly applied, maximum uniformly distributed load.

For the case of a Type 2A cover: - $\nu = 0.22$ (good estimate for polymer concrete)
$t = 0.120$ m ($r = 0 - 0.285$ m)
$t = 0.037$ m ($r = 0.285 - 0.315$ m)
$a = 0.315$ m
$F_{max} = 80$ kN (absolute maximum)
$W = 80/\pi a^2 = 256.6$ kN/m^2

The stress distribution is calculated from: -

$$\sigma = \frac{3W}{8t^2}(3+\nu)(a^2 - r^2)$$

The stress at the centre of the cover ($r = 0$ m) is estimated using $t = 0.120$ m: -

$\therefore \ \sigma = 2.13$ MPa

The stress at the lip section ($r = 0.285$ m) is estimated using $t = 0.037$ m: -

$\therefore \ \sigma = 4.07$ MPa

The high rate at which the load is applied in service effectively doubles the induced stress from the static case. The stress in the lip section at high loading rates is then: -

$\sigma = 8.15$ MPa

Finally, the effect of the small radius under the lip section is estimated to increase this stress by a factor of about 2.5. The theoretical stress in the lip section due to a suddenly applied impulse load of 80 kN is then: -

$\sigma_{max} = 20.37$ MPa

Type 2A manhole cover under SABS testing conditions

Conditions: - In this case covers are considered to be simply supported around the
edges, subjected to a statically applied, maximum central load.

For the case of a Type 2A cover: - $\nu = 0.22$ (estimate for polymer concrete)
$t = 0.120$ ($r = 0 - 0.285$ m)
$t = 0.037$ ($r = 0.285 - 0.315$ m)
$a = 0.315$ m
$F_{max} = 135$ kN

For the case of $r \neq 0$, the stress distribution is calculated from: -

$$\sigma = \frac{3(1+\nu)}{2\pi t^2} F \left(\ln \frac{a}{r} \right)$$

The stress in the centre of the plate is estimated using $r = r_e$, where r_e is a so-called *equivalent radius* given by[18]: -

$$r_e = \sqrt{1.6 r_c^{\,2} + t^2} - 0.675t$$

and r_c is the radius of an arbitrarily small area in the centre of the plate where the stress is to be determined. These methods are thus approximations, but can be used to deal with the problem. For example, if r_c is chosen to be 0.001 m, then r_e is found to be 0.039 m. This value is then substituted into the above equation to find σ at $r \approx 0$ using $t = 0.120$ m: -

$\therefore \sigma = 11.44$ MPa

The stress at the lip section ($r = 0.285$ m) is estimated using $t = 0.037$ m: -

$\therefore \sigma = 5.75$ MPa

The load is statically applied; thus there are no induced higher stresses in the component. The effect of the small radius in the lip section concentrates the stress and increases it by a factor of about 2.5. The theoretical maximum stress in the lip section is then: -

$\sigma_{\max} = 14.37$ MPa

The modulus of rupture of the material has been found to be 24.8 MPa

The stress in the lip region during the SABS testing procedure is found to be less than the stress generated in service during heavy impulse loading of 80 kN. For this reason products may pass the SABS test, yet undergo failure in service.

The force required in the testing procedure to equal the highest possible stress that may be applied in service is found by equating the two conditions: -

$$\sigma_{\max} = 8.15 MPa = \frac{3(1+\nu)}{2\pi t^2} F \left(\ln \frac{a}{r} \right)$$

The stress concentration due to the small radius increases the maximum stress by a factor of about 2.5. F is then found to be about 190 kN. It should be remembered, however, that the assumed service loading condition of 80 kN is probably overestimated.

APPENDIX B : Recommendations for SABS specification

The current SABS specification[23] applies only to components manufactured from cast iron. Mouldings produced from polymer concrete are reported to comply with the strength testing procedure detailed in this specification, yet still undergo failure in service. In particular Type 2A heavy duty components have been a cause for concern. For this reason, a more demanding compliance test is recommended to ensure reliability. It is the mode of failure that has been causing problems. Both cast iron and polymer concrete can be classed as inherently brittle materials. Before continuing, the meaning of polymer concrete as referred to hereon is given: -

Definition: - Polymer concrete here referred to consists of a *polyester* resinous binder combined with rounded fine aggregates (0.1 – 1 mm nominal size), and glass fibre reinforcement (GFR).

The table below presents a comparison of some of the more important mechanical and physical properties of polymer concrete and cast iron.

	GFR Polymer Concrete	Cast Iron
Density (g/cm^3)	2.0	7.2
Failure Load^2 (N)	1850	30,000
MOR^2 (MPa)	24.8	400
$\sigma_{UTS}{}^1$ (MPa)	12 – 16	200 – 230
K_{IC} (MNm$^{-3/2}$)	1.85	6 – 10
Flexural Modulus^2 (GPa)	4.5	60
Tensile Modulus^3, E (GPa)	15 – 18	200
Poisson's Ratio, ν	0.22[4]	0.28[5]

Notes: [1] For brittle materials the fracture load is taken to be the UTS.
[2] The figures given here are specifically for the specimen configuration used.
[3] The tensile modulus is always higher than the flexural modulus.
[4,5] Quoted from Ref. 4 and 30 respectively

Measurements have established that cast iron beam specimens of the same shape as polymer concrete specimens are over 15 times stronger in flexure. However, cast iron covers are vastly overdesigned for the loads they are built to carry. Polymer concrete components of similar dimensions are certainly strong enough to withstand the loading condition applied to them ie. fracture through the middle plane can be ruled out. However, it is the formation of cracks in the lip section that are nearly always responsible for failure of these components. The reason for this is that polymer concrete has a fracture toughness of 1.85 MNm$^{-3/2}$, while the cast irons have fracture toughness values in the range 6 – 10 MNm$^{-3/2}$. These values give good estimates of the resistance to crack initiation and propagation of the two materials. Due to its lower fracture toughness, polymer concrete is far more susceptible to crack formation in the lip section (the region of highest stress).

The types of cracks mentioned propagate during repetitive loading, and it is thus suggested that a static load test (as described in SABS 558:1973) may be inappropriate to establish the resistance to crack propagation of a manhole cover. The static test gives an idea of the resistance to crack propagation, since the load applied to the covers during this test (135 kN) is well above the maximum expected load that a manhole cover may see in service. The course of thinking is thus that if the component exhibits no cracks during a 135 kN static test, then it should exhibit no cracks during repetitive loads of 60 – 80 kN. The factor of safety used here (>2), however, is dependent on the material that the component consists of. Thus a factor of safety of 2 may apply to cast iron covers. It cannot be assumed that the same factor of safety should also apply to polymer concrete covers, since this material although inherently strong enough, is more susceptible to crack formation and propagation.

It is the design of the cover itself that plays a crucial role in the reliability of the component. For the purposes of drawing up a specification to ensure this reliability, certain minimum strength and toughness values for the material should be selected. From there, reliability of a component is determined by its design. Prospective manufacturers would have to ensure that the design of their products takes into account the lower strength and toughness values of polymer concrete. It has already been established that the AV Mouldings designed Type 2A cover has performed well in service, and stress analysis of this design has revealed that the stress concentration effects of the lip section have been dramatically reduced. For this reason it is suggested that all new Type 2A covers and frames closely follow this design. The matter will inevitably be complicated by patents and copyrights if this design is adopted. However, AV Mouldings have substantial experience regarding the performance of these products and it is absolutely certain that failure of these products will not occur.

The market for Type 2A *replacement* covers is expected to increase considerably over the next few years. Indeed, it is believed that there will always be a market for these products if their reliability can be ensured. For this reason, it is suggested that two classes of Type 2A heavy duty covers be established. These are for covers and frames that are to be installed together, and for covers intended to replace missing cast iron covers (in existing cast iron frames).

1. Material compliance

Knowledge of the generic material 'polymer concrete' is at present limited to the material studied in this research. However, it is certain that this type of polymer concrete represents the best option for use in polymer concrete manhole components from every viewpoint. Its properties are now well known, and confidence may be expressed in the suitability of the material for the required applications.

It was calculated that the material had a modulus of rupture of about 25 MPa and a fracture toughness of about 1.85 $MNm^{-3/2}$. It is recommended that polymer concrete of strength and toughness less than these values be treated with caution. The following analysis gives criteria for the minimum strength and toughness requirements: -

Toughness measurements should be conducted on sampled notched beam specimens taken from random polymer concrete mix, in three point flexure. The dimensions of the beam are important to ensure that bending stresses are dominant in the material. For this reason the span to depth ratio of the beam should be in the range 6 – 8, and if possible, closer to 10. The surface finish of the specimens should be visually assessed prior to testing. In general some voids will always be present, but a smooth gelcoat should cover most of the specimen. In particular, the bottom surface of the specimens should be inspected.

The fracture toughness is calculated using $K_{1C} = \frac{6M_C a^{1/2}}{bw^2}Y$ and $M_C = \frac{FL}{4}$

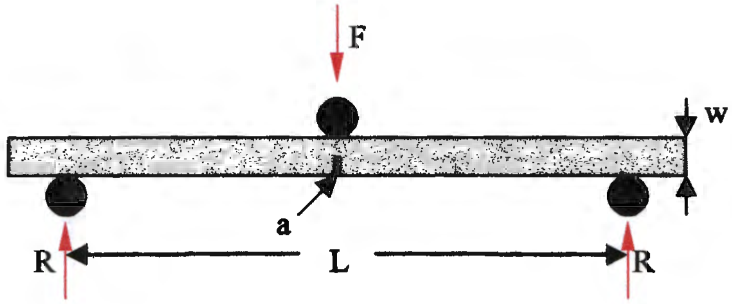
where M_C is the applied critical bending moment

a is the notch depth

b and w are the width and thickness of the specimen

$Y = 1.93 - 3.07(a/w) + 14.53(a/w)^2 - 25.11(a/w)^3 + 25.8(a/w)^4$

It is recommended that the (a/w) ratio be in the range 0.1 – 0.2.



At least ten specimens should be tested at a time. If the mean fracture toughness is less than 1.5 $MNm^{-3/2}$ then the material should be re-assessed.

For the purposes of establishing the ***strength*** of the material, similar tests should be performed. These tests are conducted in the same configuration as those for the toughness tests, but on un-notched specimens.

The ***modulus of rupture*** is found from: - $MOR = \frac{3F_C L}{2bw^2}$

where F_C is the applied critical failure load, and L is the span of the supports.

Again at least ten specimens should be tested and a mean and deviation calculated. If the MOR of the material is found to be less than 25 MPa then the material should be reassessed.

The issue of woven roving reinforcement to strengthen products is more a design consideration than a material requirement. It has been found that the positioning of the reinforcement and the impregnation with resin achieved during moulding are critical factors in determining the effectiveness of the reinforcement.

Requirements regarding workability, gel time, and health standards of wet mixes are beyond the scope of these recommendations, and at present fall at the manufacturer's discretion.

It should also be mentioned that the refinement of the properties of fibre reinforced polymer concrete is an ongoing process, and as such not too much stipulation on constituents and proportions of mix design are given. However, these strength recommendations must apply to polymer concrete in general. The process of reducing the void content of polymer concrete mix is to be experimented with. This property of polymer concrete is dependent on the type of mixing that was undertaken, and the degree of compaction achieved in the mould. It severely affects the mechanical strength of polymer concrete components.

2. Design considerations

It has been established that of all the products currently manufactured from polymer concrete, only the Type 2A replacement cover has been known to fail in service. For this reason no design analysis of other products has been undertaken, and it is (tentatively) suggested that the current design of those products is probably adequate for their intended service conditions. This assumption is made on the basis of extensive previous experience of these products in service.

For the Type 2A heavy duty covers the two categories are: -

- Type 2A heavy duty covers and frames intended for use where a new frame may be installed.
- Type 2A heavy duty covers intended to replace missing cast iron covers *in existing cast iron frames.*

The design of Category 1 covers and frames is illustrated in figure 2. Only the thickness of the lip section has been modified, and all other tolerances are the same as those for the current SABS specification for cast iron covers. Due to the lower density of polymer concrete compared to cast iron, it is recommended that the cover be cast as a solid bulk without hollows, indents, or other ribbing features in order to ensure that it's mass is sufficient to avoid it bouncing out of its frame. This requirement also ensures that changes in section do not contribute to stress concentrations in the material.

For Category 2 Type 2A replacement covers the recommended design is shown in figure 3. The cover should be cast as a bulk component to achieve minimum mass requirements. The radius under the lip section (15 mm) is a critical part of the design and should be carefully inspected after casting. These covers are inherently weaker than Category 1 covers, and their performance in service has not been well established. At present, stress analysis has revealed that the radius under the lip section significantly reduces the stress concentration in this region, but reliability of these covers will not be established without appropriate experience of the components in service.

One of the main problems in the use of these covers it that at present they do not always achieve the required fit into cast iron frames. Variations in frame size and shape (especially old frames) often result in the fact that the fitment of a polymer concrete cover into these frames becomes a matter of trial and error.

The use of reinforcements is recommended to strengthen both categories of components. It has been established that two layers of woven matting improve the performance of products in service. The positioning of the reinforcement is important, and for Category 2 replacement covers becomes critical in ensuring reliability. Two layers of reinforcement should be used in these covers; one approximately halfway through the thickness of the bulk section, and the other near the top of the bulk section extending up the side walls of the moulding and into the lip section as shown in Figure 4. At least 10 mm should be left between the side walls of the moulding and the internal reinforcement.

As has already been mentioned, good impregnation of woven matting is required to achieve these improvements. It is left to the discretion of the manufacturer on the methods he/she may use to achieve the required impregnation if these reinforcements are used. Suffice it to say that improper placement and/or impregnation of these reinforcements may have deleterious rather than beneficial effect on the mechanical properties of the components.

3. Considerations for service use

3.1 Polymer concrete components should not be used for load bearing applications where *constant* immersion in water is a requirement.

3.2 Polymer concrete components are unsuitable for load bearing applications in cases where constant *or* intermittent exposure to seawater or saline groundwater is required. Exposure to atmospheric conditions with a high salt content has not been proven to affect the mechanical properties of polymer concrete, but this factor should be assessed prior to component installation.

3.3 Polymer concrete manhole covers should not be used where intermittent or constant exposure to solutions having a pH lower than 4 or higher than 9 is required. This includes cases where the effluent flowing underneath the manhole cover has these characteristics.

These considerations apply to both Category 1 and Category 2 Type 2A covers and frames, although the effect on Category 1 covers will be less than that on Category 2 covers.

4. Construction considerations

4.1 Covers and frames should be sound and free from surface defects such as chips, scrapes, surface wet spots, excessive voids etc.

4.2 For Category 1 components, after appropriate surface finishing, covers should fit evenly into frames without rocking on the bearing surfaces. Category 2 components should also fit evenly into existing cast iron frames without rocking. All traces of grit, sand, chips, rust or any other abrasive material found on the bearing surfaces must be removed prior to the fitting of the covers into their frames.

4.3 No ribbing features or hollows should be used; the entire cover must be cast as a solid bulk.

4.4 Covers should have anti-slip projections on their top surface in the same way as for cast iron covers. The pattern of these projections does not form part of these recommendations. The manufacturer's name may be incorporated as well as any other lettering as required by the purchaser.

4.5 Category 2 components must incorporate a 15 mm radius under the lip section. It is recommended that this radius be cast with the component during manufacture rather than machined in afterwards. At present Category 1 components do not require the use of such a radius under the lip section.

4.6 Plastic lifting slots incorporated into both Category 1 and 2 covers must be tightly embedded in the bulk material, ie. any movement must be avoided.

5. Strength

It has been mentioned that the static load test as detailed in SABS 558:1973 may be inappropriate for the testing of manhole covers, however a dynamic load test, although giving a better simulation of actual loading conditions, is likely to be impractical. For this reason, the testing procedure used is likely to remain as it is at present, whatever the recommendations given here. Therefore, in order to ensure reliability of the tests, a higher test load is recommended here. In future, if funds allow, and if the demand for these products has exceeded all expectations, a complete review of the testing procedure is recommended. A general guide to a reviewed testing procedure is given here.

The static testing procedure for Type 2A polymer concrete covers at present should be as follows: -

5.1 Category 1 covers should be tested in their respective polymer concrete frames. Category 2 covers should be tested in a suitable cast iron frame.

5.2 The testing procedure for Category 1 covers is identical to that for standard Type 2A cast iron covers as described in SABS 558:1973. The testing procedure for Category 2 covers is conducted in the same way except that the applied force is increased from 135 kN to 150 kN. The rate of application of the force is kept constant, and the maximum force must be maintained for at least 1 minute. The load is then removed and the component is inspected for signs of permanent set or cracking under the lip.

A new dynamic testing procedure that has been considered involves the equipment shown schematically in figure 1. The schematics show a *general* method involved in applying a test impulse load to a manhole cover. Detailed development of such an apparatus is not considered to be one of the objectives of these recommendations.

For load measurement purposes, specially calibrated load cells would be connected to the moving parts. In addition, arrangements would have to made to prevent excessive wear of the cams and axle surfaces.

It is particularly convenient that the loading cycles (rate of repetition) and maximum applied load can be easily varied in this design.

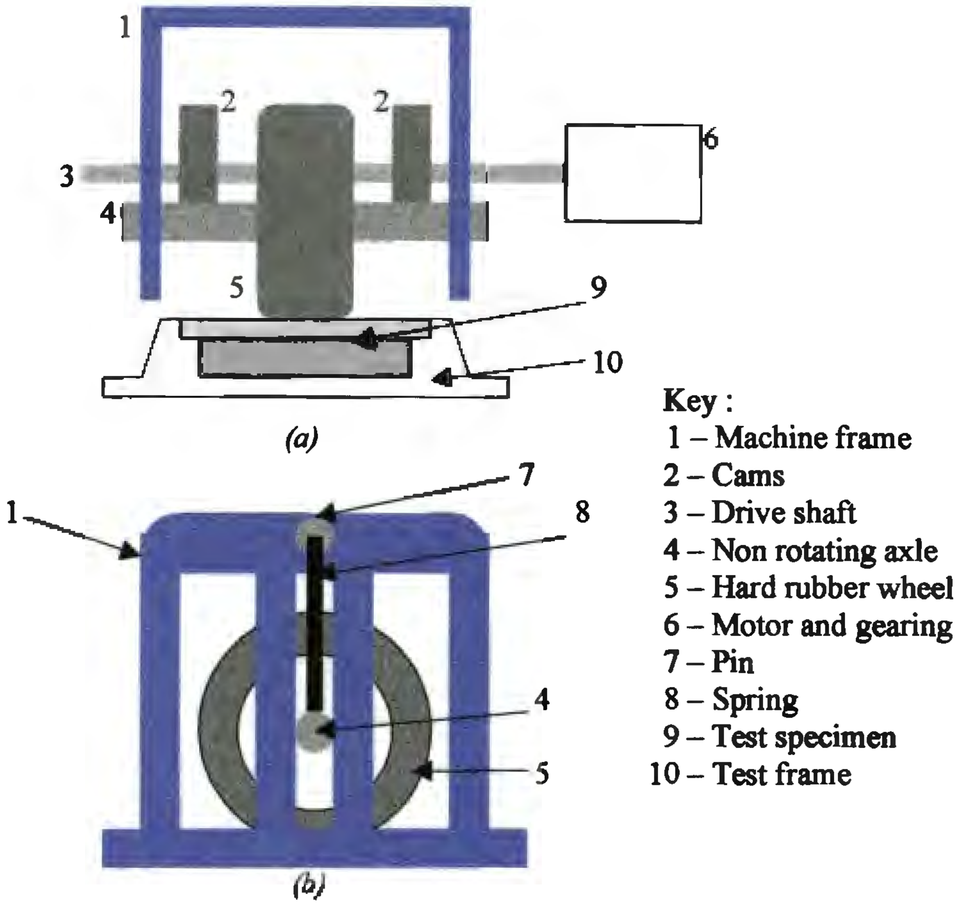


Figure 1 (a) and (b). Example of a dynamic load testing apparatus for manhole covers.

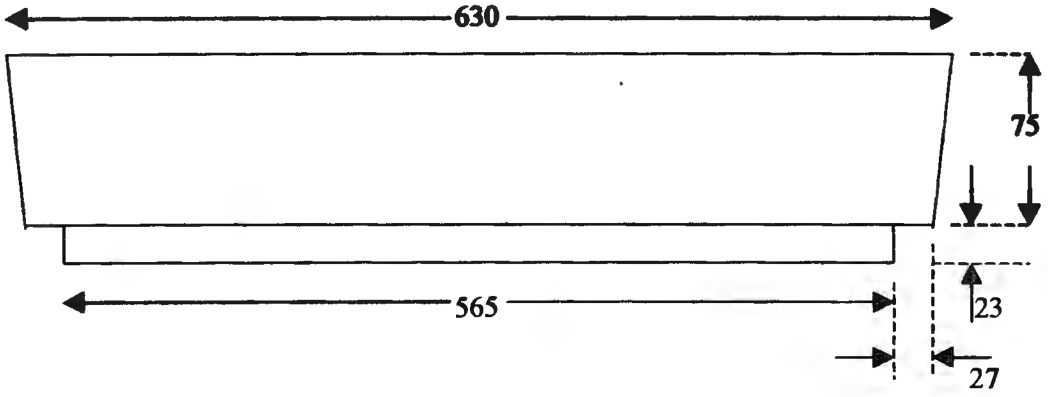


Figure 2. Proposed design for Category 1 covers. Dimensions given are approximate (in millimetres).

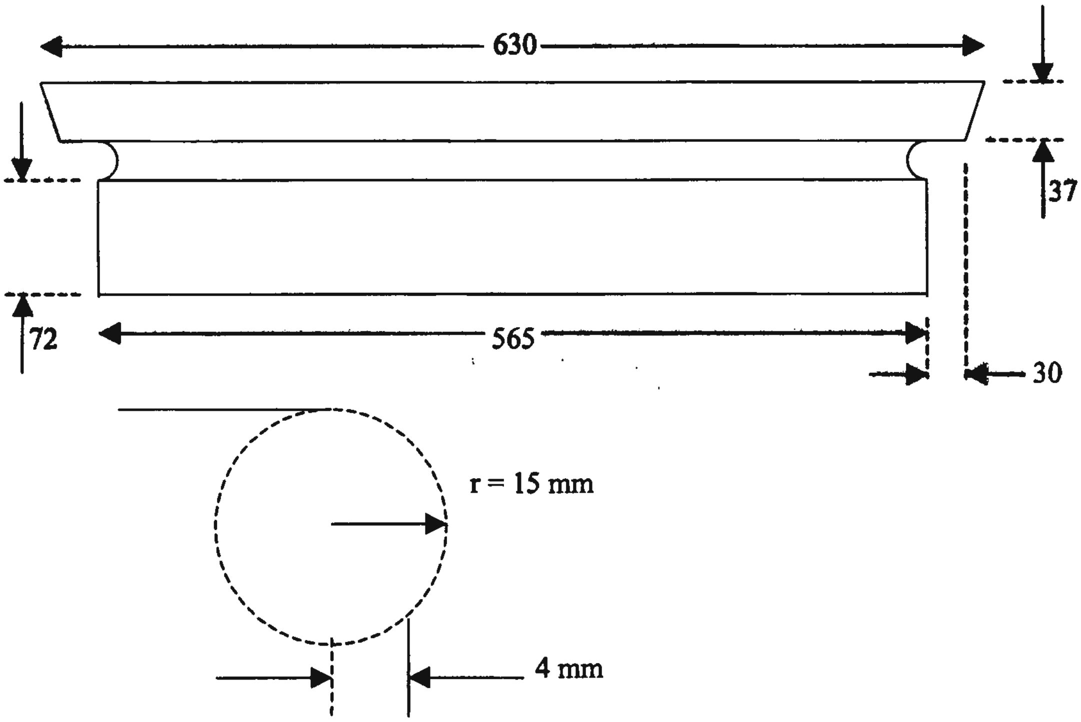


Figure 3. Proposed design for Category 2 replacement covers. Dimensions given are approximate (in millimetres).

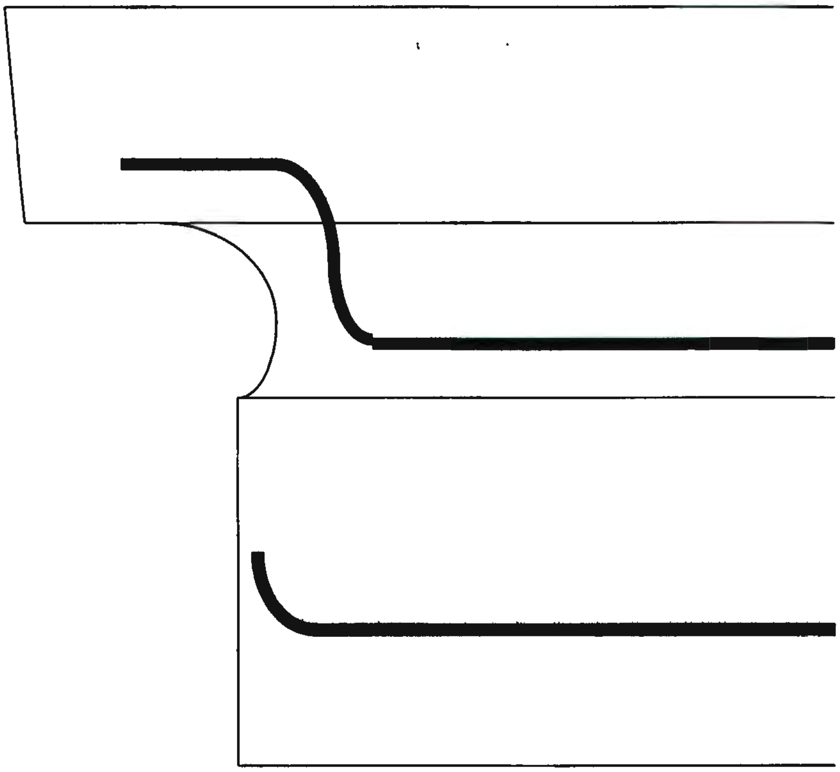
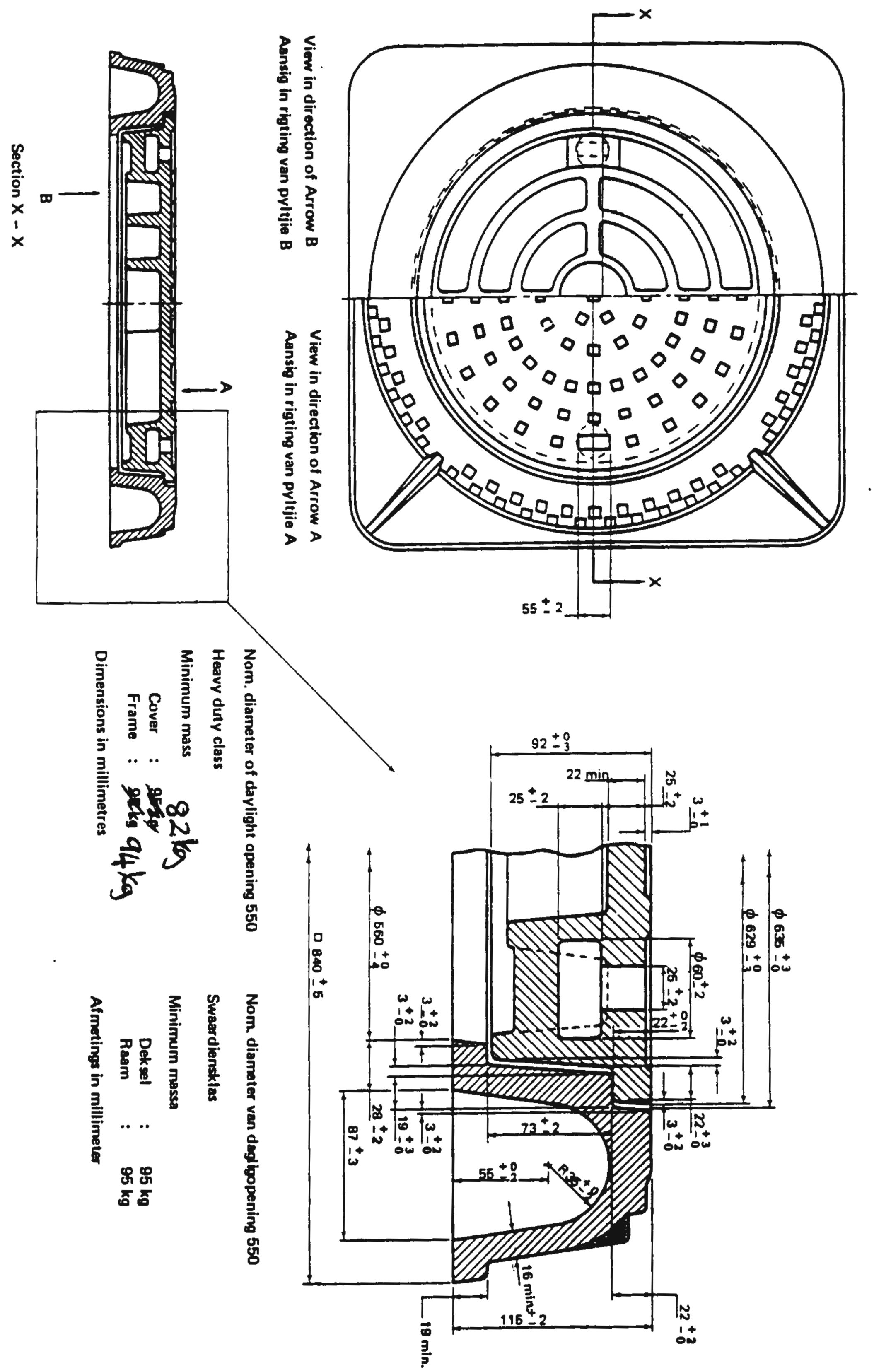


Figure 4. Schematic cross section showing the approximate positioning of woven roving reinforcement in Category 2 replacement covers.

APPENDIX C : Schematic of Type 2A manhole cover and frame specification (SABS 558 : 1973)



Nom. diameter of daylight opening 550		Nom. diameter van daglopening 550	
Heavy duty class		Swaardiensklas	
Minimum mass		Minimum massa	
Cover :	~~95 kg~~ 82kg	Deksel :	95 kg
Frame :	~~95 kg~~ 94kg	Raam :	95 kg
Dimensions in millimetres		Afmetings in millimeter	

Fig. 3 - Type 2A : Circular Manhole Cover and (Square Base) Frame, Solid Top Type
Tipe 2A : Ronde Mangatdeksel en Raam (met Vierkantige Basis), Tipe met Soliede Bovlak