Static flexural behaviour of fly ash-based geopolymer composite beam: An alternative railway sleeper

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Static Flexural Behaviour of
Fly Ash-Based Geopolymer Composite Beam:
An alternative railway sleeper

A dissertation submitted by

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B. Eng (Civil)

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Statement of Originality

‘I hereby declare that this submission is my own work and to the best of my knowledge it contains no materials previously published or written by another person, or substantial proportions of material which have been accepted for the award of any other degree or diploma at UNSW or any other educational institution, except where due acknowledgement is made in the thesis. Any contribution made to the research by others, with whom I have worked at UNSW or elsewhere, is explicitly acknowledged in the thesis. I also declare that the intellectual content of this thesis is the product of my own work, except to the extent that assistance from others in the project's design and conception or in style, presentation and linguistic expression is acknowledged.

Md. Wahid Ferdous
This thesis is dedicated to my family
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Abstract

These days, the most commonly used materials for railway sleepers are timber, concrete and steel. High maintenance cost, installation issues, deterioration of materials, and environmental considerations are among the factors that prompt research and industry to seek a different and more efficient system. This research was undertaken with the aim of developing a material for sleepers which may overcome some of these problems. Through a comprehensive survey of the literature, it was decided to propose and investigate a sleeper system composed of fly ash-based geopolymer concrete and pultruded FRP profile composites.

Although the engineering properties of fly ash-based geopolymer concrete have previously been studied, very little work has been conducted on mix design procedures that may be suitable for this new type of concrete. This study proposes a method for selecting the mix proportions of geopolymer concrete which may be suitable for concrete containing fly ash to be used as a cementitious material. A range of mixes using various water-to-geopolymer solids ratios and different amounts of fly ash content were made to test the method. The experimental results showed that certain basic principles established for conventional concrete still hold true for geopolymer concrete mix designs.

An experimental investigation was then conducted into the flexural behaviour of the composite beam made from geopolymer concrete and pultruded FRP composites which was proposed as an alternative composite sleeper. The structural analysis using applied loading showed good correlation with experimental results. Each beam was tested in a four-point bending setup to determine such characteristics as its bending modulus (E) and modulus of rupture (MOR). The proposed composite beam satisfied the minimum flexural requirements for composite railway sleepers stated in the American Railway
Engineering and Maintenance-of-way Association (AREMA) and Chicago Transit Authority (CTA) standards and also showed satisfactory performance when compared with existing railway sleepers. Finally, this study concluded that introducing this novel, environmentally friendly, composite railway sleeper to the railway industry may prove to be a viable alternative.
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\( M_C \) Moment developed at the centre of sleeper in real railway track
\( M_{C+} \) Centre positive bending moment of prestressed concrete sleeper
\( M_{C-} \) Centre negative bending moment of prestressed concrete sleeper
\( M_{R+} \) Rail seat positive bending moment of prestressed concrete sleeper
\( M_{R-} \) Rail seat negative bending moment of prestessed concrete sleeper
\( n_{gc} \) Modular ratio
\( P \) Total load applied on both load points of tested sleeper
\( P_{ab} \) Maximum contact pressure between sleeper and ballast
\( P_{the} \) Theoretical ultimate load of composite beam
\( Q \) Static wheel load
\( R \) Rail seat load
\( t \) Thickness of the pultruded composite profile
\( W \) Pressure per unit length of sleeper
\( y \) Distance of the neutral axis from the top of the beam section
\( y_{F_{p1}} \) Location of \( F_{p1} \) from the top of the section
\( y_{F_{p2}} \) Location of \( F_{p2} \) from the top of the section
\( y_{F_{p3}} \) Location of \( F_{p3} \) from the top of the section
\( y_{F_{p4}} \) Location of \( F_{p4} \) from the top of the section
\( y_{F_c} \) Location of \( F_c \) from the top of the section
\( \alpha \) A coefficient
\( \beta \) A coefficient
\( \Delta_m \) Deflection at the mid-span of composite beam
\( \varepsilon_1 \) Strain at the top fibre of composite
\( \varepsilon_2 \) Strain at the contact section between concrete and composite below neutral axis
\( \varepsilon_3 \) Strain at the bottom fibre of composite
\( \varepsilon_c \) Ultimate strain of concrete
\( \varepsilon_p \) Ultimate tensile strain of pultruded FRP composite
# Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Elaboration</th>
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<tbody>
<tr>
<td>AAR</td>
<td>Alkali aggregate reaction</td>
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<tr>
<td>AREA</td>
<td>American railway engineering association</td>
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<tr>
<td>AREMA</td>
<td>American railway engineering and maintenance-of-way association</td>
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<tr>
<td>CFRP</td>
<td>Carbon fibre reinforced plastic</td>
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<td>CTA</td>
<td>Chicago transit authority</td>
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<td>DEF</td>
<td>Delayed ettringite formation</td>
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<td>DF</td>
<td>Distribution factor</td>
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<tr>
<td>FRP</td>
<td>Fibre reinforced plastics</td>
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<tr>
<td>GCB</td>
<td>Geopolymer concrete beam</td>
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<tr>
<td>GFRP</td>
<td>Glass fibre reinforced plastic</td>
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<tr>
<td>LVDT</td>
<td>Linear variable differential transformer</td>
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<tr>
<td>MOR</td>
<td>Modulus of rupture</td>
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<tr>
<td>NCB</td>
<td>Normal concrete beam</td>
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<tr>
<td>OD</td>
<td>Oven dry</td>
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<tr>
<td>OPC</td>
<td>Ordinary portland cement</td>
</tr>
<tr>
<td>SSD</td>
<td>Saturated surface dry</td>
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<td>XRF</td>
<td>X-ray fluorescence</td>
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Chapter 1

Introduction

1.1 General

Railway transport plays a significant role in any transport system because the development of a country’s trade, industry and commerce largely depends on it. A railway tie/cross tie (in the USA), or railway sleeper (in the rest of the world) is a rectangular beam used as the base on which the rails of a railroad track are supported and fixed and is placed transversely to the rails. It is one of the most important components of a rail track, as it is positioned between its rails and ballast. The main functions of a sleeper are to transfer wheel loads from the rails to the track ballast and subgrade, and hold the rails to the correct gauge [1]. The materials commonly used for railway sleepers in rail tracks are concrete, steel and timber. However, in certain circumstances and particular environments the existing sleepers used in railway tracks did not meet the requirements satisfactorily due to their high maintenance cost, installation difficulties and durability issues [2-4]. This research is an initiative aimed at establishing a new concept for sleepers, which has a lower maintenance cost, and is easy to install in a rail track, and is environmentally friendly.

Fig. 1.1: Basic components of typical railway track system
1.2 Background to Railway Sleepers

Timber was the earliest material used for railway sleepers, from which more than 2.5 billion of sleepers worldwide were made. Timber sleepers are adaptable and have excellent electrical and sound-insulating properties. Around the 1880s, due to timber’s scarcity and the sensitivity to its use, steel railway sleepers were introduced and, currently, approximately 13% of Australia’s sleepers are made from steel. In recent years, their design has evolved and they are being replaced by the modern ‘Y’ shaped steel sleeper. During the last 50 years, the railway industry has focused on a cement-based concrete sleeper rather than the two mentioned above. The use of mono-block prestressed concrete sleepers started in 1943 and the majority of sleepers in Australian rail tracks are of this type.

The question arises as to why the railway industry uses a variety of sleeper materials rather than a particular one? Undoubtedly, one reason is that none of the existing materials (timber, steel and concrete) can satisfactorily meet the requirements of a sleeper. Therefore, currently, researchers in different parts of the world are looking for alternatives, e.g: Palomo in Spain [5] and Uhera in Japan [6] are testing geopolymer concrete; Manalo [3], Sckisui [7], Pattamaprom [8], Hoger [9], Cromberge [10], Lampo [11] and some other companies [12, 13] are trying to develop composite sleeper; and

![Different types of sleeper: (a) timber; (b) concrete; and (c) steel](Image)
other researchers [14-16] are investigating retrofitting existing timber sleepers to make them more suitable. Nonetheless, the railway industry is using mainly the three old materials instead of the recently invented composite sleeper which cannot be manufactured within the allowable cost range and has not produced results from long-term performance testing.

During the last few decades, the many advantages of fibre composites [17, 18] and fly ash-based geopolymer concrete [19-21] as suitable replacements for steel and cement concrete, respectively, have attracted the attention of researchers. Many reports [22, 23] already published on carbon-dioxide (CO2) emissions have indicated that the steel and cement industries are among the highest CO2 emitting industries. The felling of huge numbers of trees for timber sleepers as well as the production of significant amounts of CO2 by the cement (concrete sleeper) and steel (steel sleeper) industries have a negative impact on the environment. This research has been conducted during a time when world leaders are considering how to minimise carbon emissions.

1.3 Problem Statement

As previously mentioned, the primary materials used to manufacture railway sleepers are timber, concrete and steel. The problems of timber rotting, splitting and being infested with insects, as well as its scarcity, became a new challenge which led to the use of steel and concrete. Steel’s risk of corrosion, high electrical conductivity, fatigue cracking in the rail seat region and the difficulty of packing it with ballast has made it an inferior material to be used in sleepers. On the other hand, prestressed concrete sleepers, which offer greater durability than timber and steel, suffer from heavy weight,
high initial cost, low impact resistance, susceptibility to chemical attack and consequently have failed to satisfactorily meet demands [4].

These problems of existing sleeper materials have forced civil engineering material researchers to think about other possible materials. This study aims to find an alternative material for a railway sleeper which will be readily available, has a lower maintenance cost, durable, possesses low electrical conductivity, offers longer service life and, most importantly, is environmentally friendly.

1.4 Objectives of Research

To establish the concept of geopolymer filled pultruded composite beam as an effective railway sleeper it is of utmost importance to evaluate its performance. This research explores the static performance of a geopolymer concrete-based FRP composite beam with the aim of understanding its suitability as a sleeper, and its main objectives are to:

a) develop a mix design procedure for fly ash-based geopolymer concrete that is most suited for sleeper industry;

b) test and select a suitable mix of fly ash-based geopolymer concrete for a composite railway sleeper; and

c) investigate the flexural behaviour of geopolymer concrete-based composite sleepers.
1.5 Scope of Study

This study covers the feasibility of geopolymer concrete-based composite sleepers. It focuses on:

(a) a review of existing materials used in railway sleepers;

(b) discussing specific problems of existing railway sleepers;

(c) discussing the suitability of fly ash-based geopolymer concrete and pultruded FRP composites as materials for sleepers;

(d) the properties of geopolymer concrete ingredients and their characterisation with respect to the performance in railway sleepers;

(e) a detailed procedure for a fly ash-based geopolymer concrete mix design;

(f) the design method of existing railway sleepers and their flexural performance requirements;

(g) the testing and evaluation of the behaviour and failure mechanisms of a composite sleeper; and

(h) a comparison of the theoretical and experimental ultimate capacities of a composite sleeper.

Although a wide approach was taken to establish the fundamentals of a composite sleeper from which further research could continue, the following issues were beyond the scope of this research,

(a) the impact and fatigue behaviour of a composite beam;

(b) the study of its electrical impedance and thermal behaviour;

(c) its durability and environmental effects; and

(d) its finite element modelling.
1.6 Organisation of Thesis

This dissertation consists of six chapters which describe the different investigations conducted in this study.

Chapter 1 presents a general introduction to the current research, the problem statement, the objectives and scope, and the organisation of the dissertation.

Chapter 2 reviews the existing materials used for railway sleepers and their specific problems. Possible alternative solutions and arguments in support of this study’s hypothesis are discussed.

Chapter 3 is concerned with the general considerations and methods for the design of concrete sleepers, and the flexural performance requirements for timber, concrete and other existing composite sleepers according to the relevant standards.

Chapter 4 discusses the currently available mix design procedures for fly ash-based geopolymer concrete and their limitations. Illustrated using a flow chart, a method for mix design which eliminates these limitations is proposed and an example of how it works is provided.

Chapter 5 focuses on characterising the flexural behaviour of the proposed composite sleeper. The experimental investigation includes determining its failure mechanism, load-displacement behaviour, energy absorption characteristics, modulus of rupture and modulus of elasticity. Its theoretical ultimate capacity is also computed by sectional analysis and its performance compared with those of existing sleepers and relevant standards.

Chapter 6 summarises the main findings of the research, presents its conclusions and provides recommendations for future research.
Fig. 1.3: Flow diagram of the dissertation
Chapter 2

Review of Materials used in Railway Sleepers

2.1 General

This chapter provides a survey of existing materials used for railway sleepers and their specific problems. It also reviews recently developed alternative sleepers and the ongoing research being conducted into them throughout the world. Then, proposed alternative materials that have superior properties to those of existing sleeper materials and their suitability for use in sleepers are discussed.

2.2 Review of Existing Materials for Sleeper

Australia has one of the longest railway networks in the world. Millions of sleepers are manufactured every year to satisfy demand for network expansions and line upgrades. Approximately three billion sleepers are currently used in rail networks around the world [24]. The primary materials from which they are manufactured are timber, concrete and steel, all of which have some advantages and disadvantages as discussed in the following sections.

2.2.1 Timber sleepers

Timber was the first material widely used for railway sleepers. Even today, there are still many railways using timber sleepers, due to their advantages of lower cost, good resilience, ease of handling, adaptability to non-standard situations and electrical insulation [25]. Hardwood and softwood are the two major types of timber generally used as sleeper materials. However, in recent years, hardwood timber for railway sleepers has become more expensive, less available and of inferior quality compared with that previously available. On the other hand, as softwood timber sleepers are not as
effective for transmitting load, they normally used with hardwood sleepers in railway tracks. The big problem inherent in timber sleepers is their susceptibility to mechanical and biological degradation, including rotting, splitting, insect infestation, plate-cutting (abrasive damage to a sleeper due to lateral motion of its plate) and spike-pull (when a spike is gradually loosened from at sleeper) which lead to their failure. Although preservatives are used to prevent timber sleepers rotting and splitting, this protection is unable to improve their overall performances [2]. Recently environmental agencies have also become concerned about the application of chemical preservatives in timber sleepers and their proper disposal when the sleepers’ removal [26].

Fig. 2.1: Railway track with timber sleepers

2.2.2 Reinforced concrete and prestressed concrete sleepers

The advantages of concrete technology led to the use of concrete for sleepers in the 1950s. Now-a-days, approximately 500 million railway sleepers in the world’s railway networks are made from concrete and, every year, the demand for them constitutes more than 50% of total demand [24, 27]. Two types of concrete sleepers are commercially available: twin-block reinforced and mono-block prestressed. The former was originally developed in France and is used in Europe, India, Brazil and Mexico while the latter
was first developed in the UK and has been adopted in countries such as Australia, Canada, China, Japan, UK, USA and the former USSR [28].

2.2.2.1 Twin-block reinforced concrete sleeper

A twin-block reinforced concrete sleeper is made from two concrete blocks joined together by a steel tie bar cast into them. It is widely used in Europe, particularly in France. Its lower weight than a mono-block concrete sleeper is advantageous but it is difficult to handle and place due to its tendency to twist when lifted. In the early twentieth century, when concrete sleepers were first introduced into railway tracks, they failed within a few years due to brittle fracturing, cracking and their low resistance to fatigue. To overcome these weaknesses, they were reinforced by reinforcing bars which had the same lifetime as concrete and load-absorbing materials were introduced between the sleepers and the rail [29]. However, these ordinarily reinforced concrete sleepers could not provide satisfactory service [30].

![Fig. 2.2: Twin-block reinforced concrete sleepers [31, 32]](image)

2.2.2.2 Mono-block prestressed concrete sleeper

Mono-block prestressed concrete sleepers are the most commonly used sleepers throughout the world due to their greater durability in adverse environments and their resistance to twist despite twin-block reinforced concrete sleepers being more cost-effective [25]. They also distribute loads more uniformly than twin-block sleepers [29].
and, in Australia, the majority of modern railway sleepers are of this type. The first commercial prestressed concrete sleeper was developed in 1943 [33], after unsuccessful attempts at using both pure concrete and reinforced concrete, and its application started in Australia in 1970 [25]. The great advantage of prestressed concrete is that it resists tension cracks which can allow the ingress of moisture and the corrosion of embedded steel. However, the heavy weight, high initial cost, low impact resistance and susceptibility to chemical attack (delayed ettringite formation (DEF), alkali-aggregate reaction (AAR), etc.) of mono-block prestressed concrete sleepers are major problems [25, 34, 35].

Fig. 2.3: Mono-block prestressed concrete sleepers [31, 32]

2.2.3 Steel sleeper

Around the 1880s, due to the scarcity of timber and the sensitivity towards its use, steel railway sleepers, which are much stronger than timber and less expensive than prestressed concrete, were introduced. In Australia, approximately 13% of sleepers used in tracks are manufactured from steel. The advantages of steel sleepers include their light weight, ability to be stacked in compact bundles, low volume of ballast required for laying on tracks, recyclability and longer lives which have caught the attention of railway engineers [4, 29]. Conventional steel sleepers are currently being replaced by
Chapter 2

Review of materials used in railway sleepers

the recently developed ‘Y’-shaped steel sleepers which offers better support for rails [36]. However, their inverted trough profiles make them difficult to satisfactorily pack with ballast [30]. Observations of rail deflections under imposed vehicle track loadings have shown that steel sleepers deflect more greatly than timber ones, which indicates that steel and adjacent timber sleepers do not carry even proportions of an imposed wheel loading [37]. Another worrying aspect is their fatigue cracking at the rail seat region, which leads to their failure. As the good conductivity of steel sleepers creates problems for signalling, special care is required in track-circuited areas. Also, their acceptance is decreasing due to their risks of corrosion and other chemical attacks [38].

![Steel railway sleeper: (a) traditional; and (b) modern ‘Y’ shaped](image)

Fig. 2.4: Steel railway sleeper: (a) traditional; and (b) modern ‘Y’ shaped

2.3 Existing Alternative Sleeper Materials

Recently, there has been a growing tendency among civil engineering material researchers to replace existing materials for sleepers with alternative environmentally friendly substances. Some researchers [2, 14-16] have used composite materials to strengthen existing sleepers while others [7-10, 13, 39] have combined composite and other materials to manufacture new sleepers. Researchers have also taken initiatives to
develop new environmentally friendly railway sleepers using only geopolymer concrete [6, 27].

2.3.1 Retrofitting of existing sleepers

The application of a fibre composite material in a structural beam inspired several investigations into using composite materials for railway sleepers. Davalos et al. have examined at wooden cross-tie wrapped by a glass fibre-reinforced plastic (GFRP) composite. Their experimental results showed that significant increases in stiffness and strength can be achieved by wrapping a wooden cross-tie with a relatively thin layer (1.78mm) of a GFRP composite [2, 15].

In 2005, Shokrieh and Rahmat used carbon/epoxy and glass/epoxy composites to reinforce concrete sleepers. Their experimental evaluations showed that glass fibres are more effective than carbon fibres and it is possible to increase the load capacity of a sleeper by 145% when it is reinforced with glass fibres. According to them, this type of reinforcement could be a cost-effective and suitable option as it increases the fatigue life of a sleeper and prevents the growth of longitudinal cracks in it [14]. Similarly, Humphreys and Francey (2004) investigated the possibility of using carbon fibre/epoxy laminates as external reinforcements to strengthen timber railway sleepers. Their experimentation used a three-point bending test for sleepers with lengths of 2012 mm
and their manual calculations showed that the load-carrying capacity of timber sleepers can be improved using this method. However, their experimental results showed failure due to delamination [16]. Other approaches attempted to retrofit timber beam using either carbon fibre-reinforced plastic (CFRP) strips along the bottom tension layer or half-wrapping a U-shaped GFRP around the bottom of the beam. They found that increases in the moment resistance of a reinforced beams are far greater than those predicted by a simple transformed section analysis and the direct use of design code strength values [40].

Fig. 2.6: Retrofitting beams using CFRP and GFRP [40]

2.3.2 Combinations of different materials

TieTek developed new composite sleepers using recycled plastic, old tyres, waste fibreglass and structural mineral fillers which they claimed have beneficial properties over timber sleepers because they resist rail-seat abrasion and, spike pull and are not damaged by moisture, insects or fungi [11, 13]. Although TieTek sleepers have longer life-spans, they cost about two times more than concrete sleepers [8]. FFU (fibre-reinforced foamed urethane) synthetic sleepers made from hard polyurethane foam and glass fibres have been used in Japan. They have physical properties similar to timber
sleepers and are designed for more than 60 years of service life. RailCorp is the first in Australia to have used this product as a trial turnout sleeper. The use of these sleepers is increasing in situations in which maintenance and replacement are difficult [7, 25, 39, 41].

Some researchers in Thailand have investigated the possibility of using a natural rubber composite in railway sleepers in which the mechanical properties of natural rubber can be modified [8]. Using bulk recycled plastic as a material for railway sleepers was investigated by Hoger in 2000. His study concluded that, although this material increases the strength of railway sleepers, it may not be competitive in terms of cost [9]. In South Africa, polymer sleepers have been introduced as an alternative to concrete sleepers for use in the mining industry to support underground railway lines [10]. In 2002, recycled plastic composite sleepers manufactured from recycled plastic bottles combined with glass fibre reinforcement were introduced in the USA as replacements for timber sleepers. Although the manufacturer claimed that they are able to solve many drawbacks of timber sleepers, their performances in the real tracks are now being investigated [11]. Railway sleepers made from glass fibre with polyurethane foams have been used in several bridge projects in Austria and Vienna. Although they are

Fig. 2.7: FFU sleepers [7]
lightweight, and can be drilled easily using conventional tools, the initial investment cost required for their installation is comparatively higher than that for timber sleepers [7]. LLC manufactured a composite railroad sleeper from recycled high density polyethylene, recycled rubber, steel and concrete which they claimed has an improved life cycle and excellent track characteristics [12]. In 2007, Chow [42] conducted a series of tests on the static bending properties, compressive modulus of elasticity, surface hardness and three spike-resistant properties of IntegriCo composite sleepers made from composite plastics and oak. Their test results satisfied the minimum requirements of the AREMA standard. Another research study, in which the team took the initiative to develop an alternative sleeper material from glass fibre composite skins and modified phenolic foam, was conducted in the University of Southern Queensland [43]. Carrasco et al. (2012) investigated the static behaviour of glulam wood sleepers and their experimental results showed elevated performances compared with predicted formula values [44].

2.3.3 Geopolymer concrete railway sleeper

In 2007, Palomo et al. investigated the use of alkali-activated fly ash concrete in railway sleepers and suggested that it could be a suitable material for railway sleepers. However, their study did not provide adequate information regarding these sleepers’ performances [27]. Recently (2010), Uehara proposed an environmentally friendly geopolymer prestressed concrete sleeper manufactured using fly ash as the binder in concrete. To accelerate the geopolymeric reaction, a large volume of potassium hydroxide (KOH), that is commercially more expensive than other suitable alkaline liquids such as sodium hydroxide (NaOH), was used. This sleeper satisfied the static performance requirements according to the standard they used, JIS E 1202 [6]. Palomo
and Fernández-Jiménez manufactured alkali-activated fly ash monoblock prestressed sleepers for an industrial trial in 2011. Their experimental results met the requirements of both the Spanish and European codes [5].

Fig. 2.8: Ordinary (A) and geopolymer (B) prestressed concrete sleepers [6]

However, the alternative sleepers discussed above have gained limited acceptance by the railway industry due to their prohibitive costs and lack of long-term performance testing in areas such as fatigue, impact and durability.

2.4 Proposed Solutions

As mentioned earlier, the majority of railway sleepers in Australia are made from concrete. Currently, to prepare the concrete, cement is used as the main binding material although a small amount of fly ash is often used as a cement replacement to reduce the harmful effect of the alkali-aggregate reaction [25]. Since many reports on the destructive effect of cement production on the environment have been published [22, 23], concrete researchers have been looking for an alternative material and, currently, fly ash-based geopolymer concrete has proven to be a suitable replacement for cement concrete due to their excellent engineering properties [19-21].
Composite materials are currently used in many civil engineering applications, such as buildings, bridges, etc., due to their engineering properties which are superior to those of steel, concrete, aluminum and wood [45, 46]. They are formed from two or more constituent materials that provide high strength and stiffness combined with lightness. Their properties of ultra-high strength, corrosion resistance, light weight, high fatigue resistance, nonmagnetism, high impact resistance and durability make composites efficient structural materials [45]. It is possible to manufacture different shapes and sizes of composite profiles as they are man-made materials. Also, the life-cycle cost of a structure is low when it is built using composite materials [45, 47, 48].

Researchers [4, 15, 16, 49] are now thinking of using fibre composites as alternative materials for railway sleepers. Successful achievements may come by combining a fibre composite with a geopolymer concrete as both have good characteristic properties. The following paragraphs are devoted to presenting the basis for, and arguments in support of, this hypothesis.
2.5 Problems of Cement Concrete as Sleeper Material

Besides the vast number of applications of ordinary portland cement (OPC) concrete in sleepers, the following areas of concern are difficult and expensive to overcome [50].

2.5.1 Sulphate attack

Soil, groundwater and sometimes, an aggregate may contain sulfates of sodium, potassium, magnesium and calcium which, when present in a solution, can react with the tricalcium aluminate or calcium hydroxide components of cement paste. This reaction causes expansion, which leads to cracking and, finally, deterioration of the concrete [51, 52] as depicted in Fig. 2.10. The reactions representing the mechanism of this type of deterioration are:

\[
\begin{align*}
C_3A + CaSO_4.2H_2O & \rightarrow C_4ASH_{12} \text{ (monosulfate)} \rightarrow C_6AS_3H_{32} \text{ (ettringite)} \\
Ca(OH)_2 + Na_2SO_4 + 2H_2O & \rightarrow CaSO_4.2H_2O + 2NaOH \\
Ca(OH)_2 + MgSO_4 + 2H_2O & \rightarrow CaSO_4.2H_2O + Mg(OH)_2 \\
3C-S-H + 3MgSO_4 + 8H_2O & \rightarrow 3CaSO_4.2H_2O + 3Mg(OH)_2 + 2SiO_2.H_2O
\end{align*}
\]

Fig. 2.10: Effect of sulphate attack [53]
The delayed ettringite formation (DEF) due to internal sulphate attack can deteriorate concrete sleepers. Hime’s (1996) investigation confirmed that the cracking of prestressed concrete sleepers due to DEF may come after they have been in service for several years. According to him, in non-air-entrained concrete, the occurrence of DEF depends on the heat-curing temperature (above 60°C) and clinkers’ sulphate levels [34]. Similar causes of sleeper failure were found by Sahu et al. in 2004. However, in their research, they mentioned that DEF is not only dependent on the heat of the concrete’s curing temperature but also on the composition (alkalis, C3S, C3A, SO3 and MgO) and fineness of the cement [54].

Fig. 2.11: Cracking of prestressed concrete sleeper due to DEF [34]

2.5.2 Alkali attack

The difference between a sulphate and an alkali attack is that the reactive substance in the former is the cement while in the latter it is the aggregates [53]. Although the main source of alkalis in concrete is portland cement, sometimes unwashed sand contains sodium chloride which is an additional source. Admixtures (super-plasticisers) and
mixing water are also considered as internal sources of alkalis [52]. Silica-containing aggregates (e.g., chert, quartzite, opal, strained quartz crystals) could be affected by hydroxyl ions in alkaline cement solutions which may lead to destructive expansion [53], as shown in Fig. 2.12.

Reactive silica + Alkali → Alkali-silica gel

$$\text{SiO}_2 + \text{Ca(OH)}_2 + \text{H}_2\text{O} \rightarrow \text{CaH}_2\text{SiO}_4.2\text{H}_2\text{O (Alkali-silica gel)}$$

Alkali-silica gel + water = expansion

Fig. 2.12: Effect of alkali-silica reaction [53]

In 1992, Shayan et al. investigated the causes of parallel longitudinal cracking on the top surface and map cracking at the end of prestressed concrete sleepers. They examined both cracked and uncracked sleepers to determine the cause of cracking which showed that the alkali-aggregate reaction (AAR) is responsible for the failure of sleepers [35].
2.5.3 Acid attack

Concrete containing portland cement is not resistant to attack by strong acids [50, 52]. The most vulnerable cement hydrate is Ca(OH)$_2$ which converts to calcium salts when it comes into contact with an acid [51, 53]. Also, calcium silicate hydrate (C-S-H) and calcium aluminate hydrate can be attacked by acids [52, 53].

$$2\text{HX} + \text{Ca(OH)}_2 \rightarrow \text{CaX}_2 + 2\text{H}_2\text{O} \quad (\text{‘X’ is the negative ion of the acid})$$

Due to this reaction, the structure of the hardened cement is destroyed, as illustrated in Fig. 2.14 below.

![Fig. 2.14: Effect of acid attack [53]](image-url)
As is well known, industries and vehicles emit huge amounts of sulphur dioxide and nitrogen oxide into the atmosphere which are the primary cause of acid rain. This acid rain occurs not only in areas of high industrial activity and transportation loads but also a long way from industrial regions as a result of wind action. Concrete railway sleepers may also be affected by its harmful action.

2.5.4 Low rate of early strength gain

The rate of OPC hydration is slow, even when using high-temperature (60 to 70 °C) steam curing [50]. Prestressed concrete products require a compressive strength of at least 30 MPa at wire release which needs to be achieved after 8 hours of steam curing [50, 55].

2.5.5 Negligible tensile strength

Its low tensile strength (typically 1 to 3 MPa) is the greatest weakness of OPC concrete. As a result, steel reinforcing must be used to resist the tensile stresses it induces [50].

2.5.6 Environmental issues

The production of 1 tonne of cement generates approximately 1 tonne of carbon dioxide (CO₂) to the atmosphere. The cement industry is responsible for emitting 6% to 7% of the total amount of CO₂ worldwide [27]. However, this may reach nearly 10% in the near future due to the increasing development of infrastructure [56].

Fig. 2.15: CO₂ emissions from cement industry [57]
The problems of OPC concrete mentioned above have motivated researchers to consider alternative concretes for sleepers, with geopolymer concrete possibly being able to provide solutions. The argument for the suitability of this alternative is presented in the following sections.

2.6 Geopolymer Concrete

In 1978, Davidovits [58] proposed that a binder could be produced by a polymerisation process involving a reaction between alkaline liquids and compounds containing aluminium and silicon. The binders created were termed "geopolymers". Unlike ordinary portland/pozzolanic cements, geopolymers do not form calcium-silicate-hydrates (CSHs) for matrix formation and strength, but silica and alumina reacting with an alkaline solution produce an aluminosilicate gel that binds the aggregates and provides the strength of concrete. Davidovits schematically explained the geopolymerisation of an aluminosilicate oxide with a strong alkali silicate as follows:

\[
\begin{align*}
n(Si_2O_5,Al_2O_2) + 2n(SiO_2) + 4nH_2O + NaOH or KOH &\rightarrow Na^+ , K^+ + n(OH)_3-Si-O-Al-O-Si-(OH)_3 \\
(\text{Si-Al materials}) &
n(OH)_2 \\
(\text{Geopolymer backbone})
\end{align*}
\]

The last term in this formation of geopolymers indicates that water is released during the chemical reaction and is then expelled from the mixture during the curing and further drying periods, thereby enhancing the workability performances of geopolymers. This is in contrast to ordinary portland cement hydration in which water has a crucial role.
2.6.1 Constituents of geopolymer concrete

Source materials and alkaline liquids are the two main constituents of geopolymers, the strengths of which depend on the nature of the materials and the types of liquids.

2.6.1.1 Source materials

Materials containing silicon (Si) and aluminium (Al) in amorphous form, which come from natural minerals or by-product materials, could be used as source materials for geopolymers. Kaolinite, clays, etc., are included in the natural minerals group whereas fly ash, silica fume, slag, rice-husk ash, red mud, etc., are by-product materials. For the manufacture of geopolymers, the choice of source materials depends mainly on their availability and cost, the type of application and the specific demand of the users [59].

In the past, researchers have investigated several minerals and industrial by-product materials for use as source materials in geopolymer concrete including fly ash [19, 60-62], slag [63, 64], metakaolin [65], natural Al–Si minerals [66], as well as combinations of fly ash and metakaolin [67], slag and metakaolin [68], fly ash and slag [69], and calcined (fly ash) and non-calcined (kaolinite and albite) [70]. Fly ash-based geopolymer concretes provide excellent engineering properties that make them suitable materials for structural applications [62, 71].

Fly ash

Fly ash is a fine grey powder with, typically, spherical glassy particles and is produced as a by-product of coal-fired power stations. It is finer than portland cement and lime, ranging in diameter from less than 1 μm to no more than 150 μm. It has pozzolanic properties, which means that it reacts with lime to form cementitious compounds and is commonly known as a supplementary cementitious material. ASTM C618 classifies fly ashes into three major categories: Class N, Class F and Class C mainly according to the
amounts of calcium, silica, alumina and iron oxide they content. According to this standard, at least 70\% of Class N and Class F, and 50\% of Class C fly ashes are composed of pozzolanic compounds \((\text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3)\) [72].

From the consideration of calcium oxide- as a source material for geopolymer concrete, low-calcium (ASTM Class F) is preferred to high-calcium (ASTM Class C) fly ash. To produce optimal binding properties by alkali activation, the chemical composition of fly ash should have a percentage of unburned material (loss on ignition) of less than 5\%, the \text{Fe}_2\text{O}_3 content should not exceed 10\% with a low \text{CaO} content, the amount of reactive silica should be between 40-50\%, and 80-90\% of particles should be smaller than 45 \mu m [73]. The polymerisation setting rate may be hampered and the microstructure of a geopolymer changed if a high amount of calcium is present in fly ash [74]. Fortunately, in Australia, mainly Class F fly ash, which contains 80 to 85\% of silica and alumina, is produced [75]. According to Ward- "Australia's 40 power stations produce 12-13 million tonnes of fly ash a year, only a sixth of which currently goes to economic products like cement and concrete. Most of it is currently used as landfill and is both a cost and a lost opportunity. There is a lot more we can do with fly ash," [76].

**Chemical composition of fly ash**

Fly ashes are particularly rich in \text{SiO}_2, \text{Al}_2\text{O}_3 and \text{Fe}_2\text{O}_3 and also contain other oxides, such as \text{CaO}, \text{MgO}, \text{MnO}, \text{TiO}_2, \text{Na}_2\text{O}, \text{K}_2\text{O}, \text{SO}_3, \text{etc.} Typical chemical compositions of fly ash and cement are given in Table 2.1.
Table 2.1: Typical chemical analysis of fly ash and cement [77]

<table>
<thead>
<tr>
<th>Chemical constituents (as Oxide)</th>
<th>Class N Fly ash</th>
<th>Class F Fly ash</th>
<th>Class C Fly ash</th>
<th>Cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silicon dioxide (SiO₂), %</td>
<td>58.20</td>
<td>54.90</td>
<td>39.90</td>
<td>22.60</td>
</tr>
<tr>
<td>Aluminium oxide (Al₂O₃), %</td>
<td>18.40</td>
<td>25.80</td>
<td>16.70</td>
<td>4.30</td>
</tr>
<tr>
<td>Iron oxide (Fe₂O₃), %</td>
<td>9.30</td>
<td>6.90</td>
<td>5.80</td>
<td>2.40</td>
</tr>
<tr>
<td>Calcium oxide (CaO), %</td>
<td>3.30</td>
<td>8.70</td>
<td>24.30</td>
<td>64.40</td>
</tr>
<tr>
<td>Magnesium oxide (MgO), %</td>
<td>3.90</td>
<td>1.80</td>
<td>4.60</td>
<td>2.10</td>
</tr>
<tr>
<td>Sulfur trioxide (SO₃), %</td>
<td>1.10</td>
<td>0.60</td>
<td>3.30</td>
<td>2.30</td>
</tr>
<tr>
<td>Na₂O and K₂O, %</td>
<td>1.10</td>
<td>0.60</td>
<td>1.30</td>
<td>0.60</td>
</tr>
</tbody>
</table>

It is noted that fly ashes have the same kinds of oxides as cement which means that they are able to introduce materials similar to portland cement when added to a concrete mix.

2.6.1.2 Alkaline liquids

The type of alkaline liquid used plays an important role in the polymerisation process [19]. Sodium hydroxide (NaOH) with sodium silicate (Na₂SiO₃) and potassium hydroxide (KOH) with potassium silicate (K₂SiO₃) are the most common alkaline liquids used in geopolymerisation [60]. Both sodium hydroxides and potassium hydroxide have a strong base and, at room temperature, exhibit almost identical solubilities in water. They can be interchanged freely in many applications with cost being the primary difference between them as potassium hydroxide costs around three times more than sodium hydroxide when measured in tonnes.

In 2005, Fernández-Jiménez and Palomo studied the effect of an alkaline liquid on the mechanical strength of fly ash-based mortar [78]. They stated that the mechanical strength of mortar increases when waterglass (Na₂SiO₃) is added to NaOH, compared with using only NaOH. The addition of waterglass increases the Si/Al and Na/Al ratios,
resulting in increased formation of N-A-S-H (sodium aluminosilicate gel) which indicates greater strength. Hardjito and Rangan (2005) showed that the compressive strength of fly ash-based geopolymer concrete can be improved by either increasing the concentration (in molar terms) of the sodium hydroxide solution or increasing the mass ratio of the sodium silicate to sodium hydroxide solutions [60].

### 2.6.2 Benefits of geopolymer concrete over normal concrete

Geopolymer concrete has become a promising construction material for future civil engineering structures owing to its several advantages over cement concrete discussed below.

#### 2.6.2.1 Economic benefits

Rangan et al. estimated that the cost of fly ash-based geopolymer concrete may be about 10 to 30 percent less than that of OPC concrete. They also confirmed that the proper usage of one tonne of fly ash earns one carbon-credit (which is equivalent to saving one tonne of CO₂) which is a significant redemption value. In addition, the excellent technical properties of fly ash-based geopolymer concrete can reduce infrastructure maintenance cost [62].

#### 2.6.2.2 Environmental benefit

The replacement of OPC concrete by geopolymer concrete is beneficial in terms of CO₂ reduction. OPC generates 1 tonne of CO₂ to produce 1 tonne of cement while only 0.16 tonnes of CO₂ are emitted from 1 tonne of geopolymer cement [50, 79]. Therefore, it is possible to reduce CO₂ emissions from the production of a concrete binder by 84% when OPC concrete is substituted by geopolymer concrete.
2.6.2.3 Acid resistance

The resistances of geopolymer and OPC concrete in acidic media were well studied by Bakharev in 2005 and the results confirmed that geopolymer concrete has superior performance in terms of resisting acid attack [80]. Experiments by Song et al. proved that the geopolymer concrete matrix remains identical to the unaffected one after a sulphuric acid attack [81]. However, Wallah et al. concluded that geopolymer concrete may be affected by acid depending on the concentration of the acid solution [82].

2.6.2.4 Resistance to sulphate attack

In OPC concrete, sulphate ions may react with calcium hydroxide to form gypsum or with calcium aluminate hydrate to form calcium sulfoaluminate or ettringite which results in expansion, cracking and spalling in the concrete [52]. Fly ash-based geopolymer concrete has an excellent resistance to sulphate attack as the reactant calcium aluminate hydrate does not significantly exist [82].

2.6.2.5 Prevention of alkali-silica reaction

Fly ash-based geopolymer concrete is beneficial for reducing the alkali-silica reaction due to its chemical reaction between alkalis and the amorphous component in the fly ash which produces cementitious binders that increase the density of the concrete, decrease its permeability and reduce the mobility of its aggressive agent. Therefore, there is a lower possibility of an alkali-silica reaction as sufficient alkalis are not available to react with reactive silica [83]. Kupwade-Patil and Allouche reported that fly ash-based geopolymer concrete is significantly less vulnerable to an alkali-silica reaction than OPC-based concrete [84]. García-Lodeiro et al. drew similar conclusions regarding the alkali-silica reaction observing that the expansive character of the gel depends largely on the CaO content [21]. Alkali-activated fly ash cement has very high
alkali (Na) content but is low in Calcium (Ca). Fly ash itself contains alkalis but only one-sixth of them are potentially reactive [52] which indicates that, if the alkali-silica reaction takes place, it will have a much less expansive character than that normally produced in cement.

2.6.2.6 Early strength gain

Fernández-Jiménez et al. plotted variations in the compressive strengths of geopolymer and OPC concrete over time. In their study, they used two types of fly ash concrete, one activated by a NaOH solution (AAFA-N) and the other a mixture of NaOH with a Na$_2$SiO$_3$ solution (AAFA-W) for comparison. The curing times and temperatures used were, respectively, 20 hours and 85 °C for fly ash-based concrete and, for OPC concrete, 20 hours at 22 °C (OPC C22) and 20 hours at 40 °C (OPC C40). These researchers showed that alkali-activated fly ash concrete is able to achieve high compressive strength in the first few hours after alkali activation and that it is higher than that of conventional concrete, as shown in the Fig. 2.16 [71].

![Fig. 2.16: Compressive strengths of geopolymer and OPC concretes [71]](image)

It can be seen in this figure that the strength of geopolymer concrete continuously increases with time but, after the first 24 hours, the rate of gain becomes slow.
2.6.2.7 Tensile strength of geopolymer concrete

Hardjito and Rangan studied the indirect tensile strength of fly ash-based geopolymer concrete. They concluded that their measured values were higher than those recommended in AS 3600-2001 [60].

2.6.3 Factors affecting properties of geopolymer concrete

Several factors identified as being important parameters which affect the properties of geopolymer concrete are discussed below.

2.6.3.1 Type of activator

The type of activator has a significant influence on the polymerisation process of geopolymer concrete. The most commonly used activator is sodium or potassium hydroxide or a combination of hydroxides with silicates (sodium or potassium silicate). Reaction occurs at a higher rate when soluble silicates are mixed with hydroxides (NaOH + Na₂SiO₃ or KOH + K₂SiO₃) than when only hydroxides (NaOH or KOH) are used [19].

2.6.3.2 Concentration of activator

Hardjito and Rangan used concentrations of sodium hydroxide solutions in the range from 8 M to 16 M. They considered the mass ratios of sodium silicate-to-sodium hydroxide solutions and alkaline liquid-to-fly ash to be between 0.4 and 2.5 and 0.30 and 0.45, respectively. Their results showed that the compressive strength of geopolymer concrete increases when the concentration of sodium hydroxide (in terms of molar) and the ratio of the sodium silicate to sodium hydroxide solutions increases [60].

2.6.3.3 Super-plasticiser

The addition of a super-plasticiser, of up to 4% of fly ash by mass, improves the workability of a fresh fly ash-based geopolymer concrete although, the compressive
strength of hardened concrete decreases slightly when the dosage is greater than 2% [60].

2.6.3.4 Water-to-geopolymer solids ratio

Lloyd and Rangan proposed a design chart for low-calcium fly ash-based geopolymer concrete based on the assumption that the aggregates are in a saturated-surface-dry (SSD) condition, that is, the coarse and fine aggregates are neither too dry to absorb water from the mixture nor too wet to add water to the mixture. However, as in practice, aggregates may contain water above the SSD condition, they suggested that this extra water must be estimated and included in the calculation of the water-to-geopolymer solids ratios given in Table 2.2 [59].

Table 2.2: Design data for low-calcium fly ash-based geopolymer concrete [59]

<table>
<thead>
<tr>
<th>Water-to-geopolymer solids ratio, by mass</th>
<th>Workability</th>
<th>Design compressive strength, (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.16</td>
<td>Very stiff</td>
<td>60</td>
</tr>
<tr>
<td>0.18</td>
<td>Stiff</td>
<td>50</td>
</tr>
<tr>
<td>0.20</td>
<td>Moderate</td>
<td>40</td>
</tr>
<tr>
<td>0.22</td>
<td>High</td>
<td>35</td>
</tr>
<tr>
<td>0.24</td>
<td>High</td>
<td>30</td>
</tr>
</tbody>
</table>

2.6.3.5 Oxide molar ratio

In 1982, Davidovits stated that the ranges of the oxide molar ratios suitable for producing geopolymers may be $0.2 < \text{Na}_2\text{O}/\text{SiO}_2 < 0.28$; $3.5 < \text{SiO}_2/\text{Al}_2\text{O}_3 < 4.5$; $15 < \text{H}_2\text{O}/\text{Na}_2\text{O} < 17.5$; and $0.8 < \text{Na}_2\text{O}/\text{Al}_2\text{O}_3 < 1.20$ [85]. The compressive strength decreases with increases in both the $\text{H}_2\text{O}$-to-$\text{Na}_2\text{O}$ molar and water-to-geopolymer solids ratios but the ratio of $\text{Na}_2\text{O}$-to-$\text{SiO}_2$ has no significant effect on compressive strength [60].
2.6.3.6 **Handling time after mixing**

The handling time for the mixture of fly ash-based geopolymer concrete could be prolonged up to 120 minutes after mixing without any degradation of compressive strength [60].

2.6.3.7 **Curing temperature**

The mechanical strength of fly ash-based geopolymer concrete increases with increases in the curing temperature from 30°C to 90°C but a curing temperature beyond 60°C causes only a minimal increase [60].

2.6.3.8 **Curing time**

Hardjito and Rangan studied the effect of curing times from 4 to 96 hours on fly ash-based geopolymer concrete. Their results proved that compressive strength increases rapidly with increases in the curing time up to 24 hours beyond which the rate of strength gain is only moderate [60].

![Fig. 2.17: Effect of curing time on compressive strength][1]

**2.6.3.9 Rest period prior to curing**

The compressive strength increases with increases in the number of rest periods (time between casting and the commencement of curing) up to the first three days but, beyond this, has no significant effect [60].

---

[1]: image.png
2.7 Composite Materials

Composite materials (often shortened to “composites”) are man-made materials manufactured from two or more different constituent materials, with the resulting composite being different in physical and chemical properties from the original constituent materials [45]. One constituent is called the reinforcing or fibre phase that generally provides strength while that in which the fibre is embedded is called the matrix phase which is normally a cured resin-like epoxy that acts as a binder and holds the fibres in their intended positions.

2.7.1 Fibre-reinforced polymer composites

Fibre-reinforced polymers are sometimes called fibre-reinforced plastics or FRPs and are composite materials made from fibres, polymers and additives. The fibres are generally fibreglass, carbon or aramid while the polymer is usually an epoxy, vinylester or polyester thermosetting plastic. Their additives include plasticisers, impact modifiers, heat stabilisers, antioxidants, light stabilisers, flame retardants, blowing agents, antistatic agents, coupling agents and sometimes, small quantities of coatings, pigments and fillers are other constituents [45].

2.7.1.1 Fibres

Fibres are the principal load-carrying constituents of composites and occupy the largest volume in a composite laminate. For industrial and commercial applications, a variety of fibres are used and their arrangement in a composite depends on the structural requirements and fabrication process. Commonly used unidirectional fibres are produced in the form of single layers of yarn. Fibres are strong in tension, with their highest strength along the longitudinal direction and their lowest in the transverse or radial direction [45]. Commonly used fibres are glass, carbon and aramid [86].
Glass fibres

Glass is the most commonly used fibre for producing FRP composites. It is mainly silica-based but contains other oxides, such as calcium, boron, sodium, aluminium and iron. Several types of glass fibres, E-glass, S-glass, C-glass, D-glass and A-glass, are commercially available, with the first two generally considered to be the main ones [45, 87].

**Advantages of glass fibres**

Glass fibres are mostly used because they offer the following advantages [45, 87, 88]

a) low-priced;

b) high strength;

c) good chemical resistance; and

d) excellent insulating properties.

**Drawbacks of glass fibres**

The drawbacks of glass fibres include [45, 87, 88]

a) low elastic modulus;

b) high specific gravity;

c) sensitivity to abrasion (reduces tensile strength);

d) high level of hardness; and

e) relatively low fatigue strength.

2.7.1.2 Polymers

Polymers (also called resins or plastics) are organic compounds formed by carbon and hydrogen (e.g., methane CH₄) [45]. They can be in a solid or liquid state, and a cured polymer is called a matrix. To keep the fibres in place, transfer stresses between them, provide a barrier against an adverse environment and protect their surfaces from
mechanical degradation (e.g., by abrasion) which are the main functions of the matrix in a fibre-reinforced composite. Polymers are classified into two broad categories: thermoplastics and thermosets (or thermostetting).

2.7.1.3 Additives

To modify their material properties, additives are added to composite materials during their manufacturing process. Common additives generally used to manufacture FRP composites include plasticisers, impact modifiers, heat stabilisers, antioxidants, light stabilisers, flame retardants, blowing agents, antistatic agents, coupling agents and others [45].

2.7.2 Pultrusion process for manufacturing composites

Pultrusion is an automated process for the manufacture of straight or curved profiles, each with a constant cross-section, from composite materials. Generally, “I” “L” and “T” rectangular and circular sections, and hollow rectangular and circular tubes are available commercially [45]. The word “pul” from the pulling force applied to the fibres combined with the word “trusion” from the extrusion process of hot molten material forms the name “pultrusion”. This fabrication of composites has become increasingly popular because it is the fastest and most cost-effective process [45, 89, 90], and also produces good-quality products [17]. The average manufacturing output from the pultrusion process is about 1 to 5 linear ft/min [91].
2.7.2.1 Fabrication of pultruded FRP

The pultrusion process is based on the collection of fibres in the form of roving, tow, mat or fabric through a resin bath and then through a heated die to cure the resin. The reinforcement supplier, impregnation of the reinforcement with liquid resin, preforming, the consolidation die, the pulling system and the sawing unit are several successive units in the pultrusion process [17] which produces a continuous prismatic section similar to a pultrusion die. A variety of resin and fibre types is used to produce products ranging from simple round bars to complex architectural mouldings. To cut the continuous composite product to the desired length, a flying cut-off saw is programmed. As the transverse properties of pultruded composites are relatively poor, their fibres are oriented in the longitudinal direction [90]. This limitation can be overcome by incorporating mats and fabrics with transverse reinforcement to balance the properties [92].

Fig. 2.18: Fibreglass composite structural elements formed by pultrusion [90]
2.7.2.2 Characteristic properties of pultruded FRP

A structural engineer can take advantage of many interesting characteristic properties of pultruded FRPs, including [17, 18, 93]:

a) corrosion resistance in highly aggressive environments;
b) higher strength-to-weight ratios than steel;
c) excellent thermal and electrical non-conductivity;
d) anti-magnetic and spark-free capabilities;
e) low maintenance requirements; and
f) easy and quick on-site cutting, machining and assembling.

2.7.2.3 Typical applications of pultruded FRP

Engineers have incorporated pultruded FRPs in a wide range of applications, such as [17, 93]:

a) buildings and bridges: structural beams, coverings, ladders, hand railings, security fencing, scaffolding, etc;
b) marine: sea-wall protection units, walkways, etc;
c) mining: structural supports, oil sucker rods, etc; and
d) sewerage and water supplies: tank bracing, sluice gate guides, etc.
2.7.2.4 Advantages and disadvantages of pultrusion process

The major advantages of the pultrusion process are [90];

a) low production cost;

b) low raw material costs;

c) uncomplicated machinery; and

d) high degree of automation.

However, it has some disadvantages, including [90];

a) limited cross-sectional shapes;

b) labour-intensive setup times and initial process startup;

c) possibility of higher void contents in some parts than allowable limits; and

d) majority of reinforcement oriented in longitudinal direction.

2.7.3 Durability of FRP composite

The durability of a material or structure is defined as “its ability to resist cracking, oxidation, chemical degradation, delamination, wear, and/or the effects of foreign object damage for a specified period of time, under appropriate load conditions and specified environmental conditions” [94]. The manufacturing techniques, and fibre and resin types used play an important role in the durability of FRP composites. The possibility of a high void content during manufacturing may accelerate moisture absorption which eventually leads to degradation in both strength and stiffness [45]. It can be anticipated that a structure made from FRP composites may come into contact with atmospheric humidity, acid rain, chemicals and an alkaline environment that may lead to some micro-structural and morphological transformations during its service life. Some of the environmental conditions including chemical solutions (acids, salts and alkalines),
elevated temperatures, thermal cycling (freeze-thaw) and fatigue may affect the FRP composite’s durability [45].

2.7.3.1 Acid effect

There is little published information on the effects of acid on FRP materials. However, under acidic conditions, the deterioration of the concrete is likely to be of greater concern than the deterioration of the FRP materials [95]. Acid attack leads to a leaching process that reacts more slowly with glass than with alkalis [96].

\[
\text{Na}^+ + \text{HCl} \rightarrow \text{H}^+ + \text{NaCl}
\]

2.7.3.2 Salt effect

FRPs have excellent resistance to aqueous solutions (salts, acids, etc.) although some chemicals can attack them [97]. The rate of degradation of glass FRP composites in a salty environment follows the simple relationship \( \sigma_t = \sigma_0 e^{-\lambda t} \), where \( t < 450 \) days, \( \lambda = 0.0015 \), \( \sigma_0 = \) tensile strength of FRP at time \( t = 0 \) and \( \sigma_t = \) tensile strength of FRP at time \( t \) [98]. From this equation, it can be concluded that the maximum reduction in the tensile strength of a glass FRP is not more than 50% in 450 days after which it does not change considerably.

2.7.3.3 Alkaline effect

FRP composites can come in contact with alkaline media through interactions with a variety of sources, including alkaline chemicals, soil (or solutions diffusing through soil) and concrete. Reductions in the mechanical properties of glass FRP composites due to alkaline attack on the glass are attributed to: (1) etching; and (2) hydroxylation and dissolution. During etching, the constituents of the glass are released because the silica network is attacked by alkali.

\[
2 \times \text{NaOH} + (\text{SiO}_2)_x \rightarrow \text{X Na}_2\text{SiO}_3 + \text{X H}_2\text{O}
\]
Hydroxylation is associated with dissolution which is characterised by the leaching of calcium from the glass. Calcium hydroxide is deposited on the surface of the glass and reduces the rate of reaction when the leached calcium is combined with water [45].

2.7.3.4 Temperature effect

Sirimanna et al. [99] studied the effect of temperature on the properties of pultruded FRP composites. Their experimental results showed that the flexural strength of this material decreases linearly to approximately 28% of its initial strength when the temperature increases from 20°C to 105°C. However, decreasing the temperature may increase the elastic modulus, and tensile, flexural and fatigue strengths of a fibre-reinforced polymer [45].

2.8 Conclusions

The unexpected deterioration of existing sleepers under different conditions, as well as concern regarding the huge amount of CO₂ emitted into the environment by the cement industry, are the main drivers for this research which looks at alternative materials for railway sleepers. Research and development are now focussed on the alternatives of geopolymer concrete and pultruded fibre composites as these two materials are jointly able to overcome many weaknesses of existing railway sleepers.

As discussed earlier, geopolymer concrete has excellent engineering properties and may be 10 to 30 percent cheaper than OPC concrete. It is not only superior in the economic sense but also from the environmental point of view. Around 84% of CO₂ emissions could be prevented from being emitted if OPC concrete was substituted by geopolymer concrete.
Additionally, pultruded glass fibre-reinforced composites offer low-cost, good-quality products with excellent technological properties. The report published in the relevant area [100] suggested that, alkali resistant glass fibre can significantly improve the performance of FRP in highly alkaline environments. The FRP composed of vinyl-ester resins has superior resistance to alkalinity ingress compared to other common resins such as epoxies and polyesters.

These many advantages of geopolymer concrete and pultruded glass fibre composites strongly support the hypothesis that it is viable to develop railway sleepers which have low weights, high strengths and are more durable.

In Chapter 3, current sleeper design methods and the flexibility performance requirements for sleepers are discussed.
3.1 General

In this chapter, a thorough review of a conventional sleeper design method and the flexibility performance requirements for different sleepers are described. A wide range of sleeper design codes, including the AREMA manual, Euro Codes, UIC Leaflets, Australian Standard, RailCorp Engineering Standard, South African Railway Codes and Indian Standard, are available worldwide. A detailed procedure of sleeper design method is discussed here according to Australian Standard (AS 1085.14 - 2003). This chapter is one of the major tasks undertaken in this research in order to judge the outcome of the experimental program. These data also assist to explain the experimental results for composite sleepers.

3.2 Considerations for Sleeper Design

Sleepers should be designed to fulfil the requirements of transmitting vertical, lateral and longitudinal static and dynamic loads from rail to ballast without any disturbances of track geometry beyond permitted tolerances. Regarding precast concrete railway track systems, FIB Bulletin 37 considers the following three separate loading cases for the design of sleepers [24].

a) There should be no cracks in the sleeper when it is subjected to design wheel loads.

b) Occasionally, sleepers will be subjected to exceptional wheel loads. In these cases cracks should not exceed a small defined width after removal of a load.
which is a specified factor greater than the design wheel load. This is to ensure the sleepers remain serviceable even when subjected to these exceptional loads.

c) Failure does not occur if the sleeper is subjected to high accidental impact loads. This “factor of safety” should be specified.

They also include the following considerations when designing and detailing sleepers.

a) Sleeper sizes, minimum weight requirements and the maximum allowable pressure between the sleeper and the ballast.

b) A minimum concrete cover of steel to ensure maximum durability.

c) Adequate electrical resistance.

d) Extreme climatic conditions.

For other pre-cast concrete track systems, similar approaches will be required.

3.3 Design Requirements

3.3.1 Sleeper dimensions

RailCorp Australia [101] specified the following dimensions for railway sleepers which may change slightly depending on the medium- and heavy-duty truck types being used.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Heavy duty (axle load ≤ 30 tonne)</th>
<th>Medium duty (axle load ≤ 25 tonne)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>2390-2500 mm</td>
<td>2390-2500 mm</td>
</tr>
<tr>
<td>Width (at base)</td>
<td>220-255 mm</td>
<td>220-255 mm</td>
</tr>
<tr>
<td>Depth (centre of rail seat)</td>
<td>230 mm maximum</td>
<td>180 mm maximum</td>
</tr>
<tr>
<td>Rail seat area (flat surface)</td>
<td>28800 mm²</td>
<td>25620 mm²</td>
</tr>
</tbody>
</table>
Table 3.2: Dimensions of timber sleeper

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Dimension, (mm)</th>
<th>Acceptance tolerance, (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>2440</td>
<td>+ 50 - 0</td>
</tr>
<tr>
<td>Width</td>
<td>230</td>
<td>+ 10 - 0</td>
</tr>
<tr>
<td>Depth</td>
<td>130</td>
<td>+ 10 - 0</td>
</tr>
</tbody>
</table>

3.3.2 Gauge lengths and sleeper spacing

Gauge lengths and sleeper spacings are different in different regions of Australia. In 1973, Gordon summarised those in Australia’s railway networks [102] which indicated that gauge lengths vary from 1067 to 1600 mm and the sleeper spacings from 495 to 760 mm.

Table 3.3: Gauge lengths and spacings of sleepers in Australia

<table>
<thead>
<tr>
<th>Railway system</th>
<th>Gauge length, (mm)</th>
<th>Sleeper spacing (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>New South Wales public transport commission</td>
<td>1435</td>
<td>610</td>
</tr>
<tr>
<td>Victorian railway</td>
<td>1435 and 1600</td>
<td>625</td>
</tr>
<tr>
<td>Commonwealth railways (Australian national)</td>
<td>1435 and 1067</td>
<td>610 and 760</td>
</tr>
<tr>
<td>South Australian railways (Australian national)</td>
<td>1435, 1067 &amp; 1600</td>
<td>665</td>
</tr>
<tr>
<td>Tasmanian Railways (Australian national)</td>
<td>1067</td>
<td>610</td>
</tr>
<tr>
<td>Western Australian government railways</td>
<td>1435 and 1067</td>
<td>610</td>
</tr>
<tr>
<td>Queensland railways</td>
<td>1067</td>
<td>645</td>
</tr>
<tr>
<td>New Zealand railways</td>
<td>1067</td>
<td>610</td>
</tr>
<tr>
<td>Mount Newman Mining Co.</td>
<td>1435</td>
<td>530</td>
</tr>
</tbody>
</table>
3.3.3 Effective sleeper support area beneath rail seat

The effective sleeper support area beneath at rail seat is the product of the width of the sleeper and the assumed value of the effective length of the sleeper support at the rail seat. This effective length can be calculated by the equation of either Clarke [103] or Schramm [104].

Fig. 3.1: Principal sleeper dimensions

3.3.3.1 Clarke’s effective length and area

(a) For timber sleepers,
Clarke calculated the effective length of sleeper support $L_e$ (mm), under the rail seat using the following equation -

$$L_e = (l - g)[1 - \frac{(l-g)}{125.t^{0.75}}]$$

(3.1)

where,

$l$ = total sleeper length (mm);

$g$ = distance between the centre lines of the rail seats (mm); and

$t$ = sleeper thickness (mm).

This effective length can be approximated by

$$L_e = \frac{l}{3}$$

(3.2)

The effective sleeper support area $A_s$ (mm$^2$) beneath the rail seat is
\[ A_s = B(l - g)[1 - \frac{(l-g)}{125t^{0.75}}] \]  
(3.3)

where, \( B \) is the sleeper breadth (mm).

(b) For concrete sleepers,

There are no comparable equations for calculating the effective length of sleeper support in which the effect of sleeper thickness is incorporated.

### 3.3.3.2 Schramm’s effective length and area

Schramm calculated the effective length of sleeper support for both timber and concrete sleepers by the following equations.

\[ L_e = l - g \]  
(3.4)

\[ A_s = B(l - g) \]  
(3.5)

where the symbols are as previously defined.

### 3.3.3.3 For non-uniform sleeper

For sleepers which are not of a uniform breadth, the effective support area can be determined by assuming an average sleeper breadth over the effective length. In the case of steel sleepers with inverted trough-shaped cross-sections, the effective sleeper breadth is the maximum distance between the sleeper’s flanges.

### 3.3.4 Determination of rail seat load of concrete sleeper

The performance of a sleeper in withstanding lateral and longitudinal loading depends on its size, shape, surface geometry, weight and spacing [105]. According to Australian standard AS 1085.14, the value of the rail seat load \( (R) \) for a concrete sleeper can be calculated by

\[ R = j \times Q \times \frac{DF}{100} \]  
(3.6)
where,

\[ j = \text{combined quasistatic and dynamic design load factor}, \]

\[ Q = \text{static wheel load and} \]

\[ DF = \text{axle load distribution factor}. \]

**Combined design load factor \( (j) \)**

The quasistatic load is the sum of the static load and its effect at speed whereas the dynamic load is the load due to the high-frequency effects of the wheel/rail load interaction and track components’ responses. Therefore, the combined effect includes the static load, the allowance for the effects of static load at speed and the allowance for dynamic effects. AS 1085.14 prescribes that the value of the combined design load factor \( (j) \) shall be not less than 2.5 [55].

**Axle load distribution factor \( (DF) \)**

When the rails are equal to or heavier than 47 kg/m, the actual proportion of the vertical axle load taken by an individual sleeper for a given sleeper spacing can be obtained from Fig. 3.2.

![Fig. 3.2: Axle load distribution factor \( (DF) \) [55]](image)
3.3.5 Maximum contact pressure between sleeper and ballast

For the structural design of sleepers, it is important to know the contact pressure distribution between at sleeper and ballast and its variations with time. However, as the exact pressure distribution for a sleeper in the in-track condition is practically impossible to predict [106], basically, uniform contact pressure distribution is assumed in order to calculate the sleeper’s bending stresses. Talbot [107] assumed various hypothetical sleeper-ballast contact pressures which, together with the corresponding sleeper bending moments, are shown in the diagram in Fig. 3.3.

![Diagram showing hypothetical distribution of sleeper-ballast contact pressure and bending moment.]

**Fig. 3.3:** Hypothetical distribution of sleeper-ballast contact pressure and bending moment [107]

MW Ferdous  Static Flexural Behaviour of Fly Ash-Based Geopolymer Composite Beam: *An alternative railway sleeper*
Australian standard AS 1085.14 recommends using separate equations to calculate the maximum ballast pressures in broad and narrow gauge tracks.

For a broad gauge line \((g > 1.5 \, m)\), the maximum ballast pressure \((P_{ab})\) is

\[
P_{ab} = \frac{R}{B(1-g)} \tag{3.7}
\]

and for a narrow gauge line \((1.5 \, m > g > 1.0 \, m)\),

\[
P_{ab} = \frac{R}{0.8 \times B(1-g)} \tag{3.8}
\]

where the symbols are as previously defined and the ballast pressure does not exceed 750 kPa.

### 3.3.6 Maximum bending moment of concrete sleeper

To check the flexural performance of a sleeper under service load conditions, it is essential to know its maximum bending moment and bending stress which occur in the region of the rail seat and the centre of the sleeper [105]. Australian standard AS 1085.14 uses different pressure distribution patterns of a sleeper and ballast to calculate the maximum positive and negative bending moments at the rail seat and centre position of the sleeper. These patterns and mathematical formulations are shown in Table 3.4 and Fig. 3.4.
Table 3.4: Pressure distributions of concrete sleepers under different conditions [55]

<table>
<thead>
<tr>
<th>Moment conditions</th>
<th>Pressure distribution diagram</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| Rail seat positive bending moment for narrow gauge, $1.5m > g > 1.0m$ | ![Diagram](image1) | $a = 0.8(l - g)$  
$W = \frac{R}{0.8(l - g)}$ |
| Rail seat positive bending moment for broad gauge, $g > 1.5m$ | ![Diagram](image2) | $a = (l - g)$  
$W = \frac{R}{(l - g)}$ |
| Centre positive bending moment for broad gauge, $g > 1.5m$ | ![Diagram](image3) | $a = 0.9(l - g)$  
$W = \frac{R}{0.9(l - g)}$ |
| Centre negative bending moment for broad gauge, $g > 1.5m$ | ![Diagram](image4) | $a = (l - g)$  
$W = \frac{4R}{3l - 2g}$ |
| Centre positive bending moment for narrow gauge, $1.5m > g > 1.0m$ | ![Diagram](image5) | $a = 0.9(l - g)$  
$W = \frac{R}{0.9(l - g)}$ |
| Centre negative bending moment for narrow gauge, $1.5m > g > 1.0m$ | ![Diagram](image6) | $W = \frac{2R}{l}$ |
Fig. 3.4: Maximum bending moment of concrete sleeper according to AS 1085.14
3.4 Flexural Performance Requirements for Railway Sleeper

3.4.1 Flexural requirements for timber sleeper

The modulus of rupture (i.e., ultimate strength) for Australian timber sleepers has been quantified by Duckworth (1973) [108] and Reid (1973) [109]. Duckworth’s experimental investigations were based on treated and untreated types of timber sleepers but Reid measured the ultimate strengths for softwood and hardwood timber sleepers.

Table 3.5: Average MOR for timber sleepers measured by Duckworth

<table>
<thead>
<tr>
<th>Hardwood sleeper condition</th>
<th>Sleeper age, (years)</th>
<th>Average modulus of rupture (from 12 tests), MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry untreated</td>
<td>New</td>
<td>110</td>
</tr>
<tr>
<td>Green untreated</td>
<td>New</td>
<td>80</td>
</tr>
<tr>
<td>Green treated and incised</td>
<td>New</td>
<td>78</td>
</tr>
<tr>
<td>Dry treated</td>
<td>New</td>
<td>61</td>
</tr>
<tr>
<td>Green treated</td>
<td>New</td>
<td>47</td>
</tr>
</tbody>
</table>
Table 3.6: Average modulus of rupture for timber sleepers measured by Reid

<table>
<thead>
<tr>
<th>Sleeper type and condition</th>
<th>Sleeper age, (years)</th>
<th>Sleeper cross-section width × depth, (mm)</th>
<th>Modulus of rupture, (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Softwood</td>
<td>New</td>
<td>233 × 173</td>
<td>34</td>
</tr>
<tr>
<td>Softwood</td>
<td>New</td>
<td>230 × 170</td>
<td>22</td>
</tr>
<tr>
<td>Softwood</td>
<td>37</td>
<td>238 × 165</td>
<td>20</td>
</tr>
<tr>
<td>Softwood</td>
<td>37</td>
<td>236 × 166</td>
<td>20</td>
</tr>
<tr>
<td>Hardwood</td>
<td>New</td>
<td>257 × 133</td>
<td>55</td>
</tr>
<tr>
<td>Hardwood</td>
<td>37</td>
<td>255 × 128</td>
<td>23</td>
</tr>
<tr>
<td>Hardwood</td>
<td>37</td>
<td>267 × 127</td>
<td>19</td>
</tr>
<tr>
<td>Hardwood*</td>
<td>17</td>
<td>235 × 117</td>
<td>46</td>
</tr>
<tr>
<td>Hardwood</td>
<td>17</td>
<td>248 × 117</td>
<td>34</td>
</tr>
<tr>
<td>Hardwood**</td>
<td>17</td>
<td>251 × 114</td>
<td>23</td>
</tr>
<tr>
<td>Hardwood**</td>
<td>17</td>
<td>251 × 121</td>
<td>29</td>
</tr>
</tbody>
</table>

*one outside spike, **two outside spikes

The maximum tensile bending stress of a timber sleeper depends mainly on the type of timber used and its moisture content, and whether the sleeper has been treated with preservatives or left untreated [105]. Clarke [103] recommends that the maximum tensile bending stress for a timber sleeper should not exceed 5.5 MPa, whereas the limit recommended by the American Railway Engineering Association (AREA) is 7.6 MPa [110].

3.4.2 Flexural requirements for prestressed concrete sleeper

In 1975, AREA [111] tabulated the flexural performance requirements for prestressed mono-block concrete sleepers in terms of the required moment capacity which, at different positions of a sleeper, depends on the sleeper’s length and spacing.
Table 3.7: Flexural performance requirements for prestressed mono-block concrete sleepers

<table>
<thead>
<tr>
<th>Length, (m)</th>
<th>Spacing, (mm)</th>
<th>Required flexural capacity, (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Rail seat, (+ve)</td>
</tr>
<tr>
<td>2.44</td>
<td>533</td>
<td>24.9</td>
</tr>
<tr>
<td></td>
<td>610</td>
<td>24.9</td>
</tr>
<tr>
<td></td>
<td>686</td>
<td>24.9</td>
</tr>
<tr>
<td></td>
<td>762</td>
<td>24.9</td>
</tr>
<tr>
<td>2.51</td>
<td>533</td>
<td>25.4</td>
</tr>
<tr>
<td></td>
<td>610</td>
<td>26.6</td>
</tr>
<tr>
<td></td>
<td>686</td>
<td>28.3</td>
</tr>
<tr>
<td></td>
<td>762</td>
<td>29.4</td>
</tr>
<tr>
<td>2.59</td>
<td>533</td>
<td>25.4</td>
</tr>
<tr>
<td></td>
<td>610</td>
<td>28.3</td>
</tr>
<tr>
<td></td>
<td>686</td>
<td>31.1</td>
</tr>
<tr>
<td></td>
<td>762</td>
<td>33.9</td>
</tr>
<tr>
<td>2.67</td>
<td>533</td>
<td>28.3</td>
</tr>
<tr>
<td></td>
<td>610</td>
<td>31.1</td>
</tr>
<tr>
<td></td>
<td>686</td>
<td>33.9</td>
</tr>
<tr>
<td></td>
<td>762</td>
<td>36.7</td>
</tr>
<tr>
<td>2.74</td>
<td>533</td>
<td>31.1</td>
</tr>
<tr>
<td></td>
<td>610</td>
<td>33.9</td>
</tr>
<tr>
<td></td>
<td>686</td>
<td>36.7</td>
</tr>
<tr>
<td></td>
<td>762</td>
<td>39.6</td>
</tr>
</tbody>
</table>

Australian standard AS 1085.14 suggests investigating the bending capacities of concrete sleeper at both positions of centre and rail seat separately. Due to the limitation of time, only central bending capacities are investigated. To apply a 40 kN-m moment
at the support, ballast loads need to be applied as a uniformly distributed loading which is not possible with the available testing facilities.

### 3.4.3 Flexural requirements for composite sleeper

The minimum performance requirements for composite sleepers have been specified by the American Railway Engineering and Maintenance-of-way Association (AREMA) and Chicago Transit Authority. In 2003, AREMA added Part V to their Code under the title “Engineered Composite Tie” after the Chicago Transit Authority (CTA) had released composite plastic railroad tie bearing no CTA 1117-020 in 2002.

The requirements in these standards and the performances of some other commercially available composite sleepers are listed in Table 3.8.

<table>
<thead>
<tr>
<th>Specification</th>
<th>Modulus of rupture, (MPa)</th>
<th>Elastic modulus, (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AREMA standards</td>
<td>13.8 (min)</td>
<td>1170 (min)</td>
</tr>
<tr>
<td>CTA standards</td>
<td>17.2 (min)</td>
<td>1380 (min)</td>
</tr>
<tr>
<td>Dynamic composite LCC, [12]</td>
<td>15.2 to 17.9</td>
<td>1380 to 1725</td>
</tr>
<tr>
<td>IntegriCo sleepers, [42]</td>
<td>24.2 (Ave.)</td>
<td>2000 (Ave.)</td>
</tr>
<tr>
<td>Tietek™ sleepers, [112]</td>
<td>13.8 to 17.2</td>
<td>1200 to 1720</td>
</tr>
<tr>
<td>Polywood sleepers, [112]</td>
<td>20.7</td>
<td>1380</td>
</tr>
<tr>
<td>FFU synthetic sleeper, [41]</td>
<td>142</td>
<td>8100</td>
</tr>
</tbody>
</table>

However, Ticoalu et al. (2008) [49] investigated eight full-size timber railway sleepers using a four-point bending test arrangement with a view to selecting a suitable stiffness and modulus of elasticity for the design of fibre composite railway sleepers. Based on their statistical analysis, they proposed that the design modulus of elasticity in flexure for fibre composite railway sleepers should be 12000 MPa.
3.5 Conclusions

A detailed description of a current track design approach, especially for concrete sleepers, has been discussed according to AS 1085.14. The flexural performances of timber sleepers in terms of their modulus of rupture can vary in a large range (22 to 110 MPa) depending on the application of chemical preservatives and timber types. On average the minimum required bending capacity at the centre of a prestressed concrete sleeper should be 23 kN-m although this requirements can be changed slightly depending on its length and spacing. Recently, different companies have developed their own products as alternatives to existing sleepers, the flexural performances of which have also discussed.

In Chapter 4, the properties of its materials and a proposed method for the design of geopolymer concrete are discussed.
Chapter 4

Preparation and Design of Geopolymer Concrete for Composite Sleepers

4.1 General

The engineering properties of fly ash-based geopolymer concrete have been studied previously but very little work has been conducted on mix design procedures that may be suitable for this new type of concrete. To compare the performance of geopolymer with normal concrete, firstly, it was decided to establish a suitable mix design method for geopolymer concrete. This chapter proposes a method for selecting the mix proportions of geopolymer concrete which may be suitable for concrete containing fly ash as a cementitious material and is one of the major contributions of this research. Firstly, this chapter describes the procedure in general, illustrates it using a flowchart and then provides an example of how it works. The experimental results showed that certain basic principles established for conventional concrete still hold true for geopolymer concrete mix designs.

4.2 Critical Discussion of Currently Available Mix Design Procedure

To date, there has been very limited research on the mix design of geopolymer concrete, let alone directives on a practical and systematic procedure that takes into consideration the strength and durability of the final product. In 2008, Lloyd and Rangan [59, 113, 114] proposed a method of mix design for fly ash-based geopolymer concrete but their method did not teach how to deal with the effects on the mix design of the ingredients’ specific gravity nor did it consider the effects of the volume of air content. The
following sections discuss the limitations of the currently available mix design procedure suggested by Lloyd and Rangan and the benefits of the proposed design method over it.

**Density of concrete**

A constant concrete density of 2400 kg/m$^3$ was assumed by Lloyd and Rangan which is not realistic because the density of concrete varies from one mix to another depending on the amount of ingredients in the mix. The design method proposed in this research is not based on a fixed concrete density of 2400 kg/m$^3$.

**Specific gravities of materials**

The basic condition of a concrete mix design is that the total volume of the concrete materials calculated for 1 m$^3$ of concrete should be one cubic meter. This condition is completely dependent on the specific gravities of the materials which unfortunately, were not taken into account in the currently available mix design procedure. The proposed design method has extensively considered the ingredients’ specific gravities.

**Flexibility to improve workability**

Sometimes, extra alkaline liquid or a super-plasticiser is needed to improve the workability of fresh geopolymer concrete which has an effect on the total volume of the concrete. Their method did not explore the design for workability which in geopolymer concrete seems to assume a yet more important effect than other types of concrete. The new design method accommodates for the need to design for a suitable workability.

**Volume of entrapped air**

The volume of entrapped air cannot be ignored. It may vary from 1 to 5 percent or sometimes more but was not considered in Lloyd and Rangan’s design method. It is carefully taken into account in the design method proposed in this research.
Opportunity to use aggregates in field condition

The method described in this chapter makes it easy to account for the aggregates’ moisture conditions as they are incorporated in the spreadsheet rather than being left to an individual to calculate. Thus, the proposed mix design procedure provides flexibility and ease by using aggregates in their ‘as is’ conditions.

4.3 General Description of Proposed Mix Design Procedure

From the above considerations, it is believed necessary that a rigorous, but still easy, method for geopolymer concrete mix design be established. The following sections present such a method. Firstly, its procedure described in general using a flowchart and then a detailed example is presented.

![Flow chart for design of fly ash based geopolymer concrete](image-url)

Fig. 4.1: Flow chart for design of fly ash based geopolymer concrete

MW Ferdous Static Flexural Behaviour of Fly Ash-Based Geopolymer Composite Beam: An alternative railway sleeper
4.4 Material Properties

To minimise changes in their properties, the concrete materials were obtained from the same sources every time. The properties of concrete materials necessary for concrete mix design were determined in the laboratory.

4.4.1 Aggregates

The quality of aggregate is considered importantly in order to their three-quarters volume of concrete. The strength, durability and structural performance of concrete are greatly influenced by the properties of aggregates.

4.4.1.1 Coarse aggregate

Three different sizes of coarse aggregates (14 mm, 10 mm and 7 mm) obtained in crushed rock form were used to prepare concrete in the laboratory. The specific gravity and absorption capacities were determined according to ASTM C127.

Table 4.1: Properties of coarse aggregates

<table>
<thead>
<tr>
<th>Properties of aggregates</th>
<th>Size of aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>14 mm</td>
</tr>
<tr>
<td>Specific gravity, (oven dry)</td>
<td>2.65</td>
</tr>
<tr>
<td>Specific gravity, (saturated surface dry)</td>
<td>2.67</td>
</tr>
<tr>
<td>Apparent specific gravity</td>
<td>2.70</td>
</tr>
<tr>
<td>Absorption capacity, (% of oven-dry weight)</td>
<td>0.675</td>
</tr>
<tr>
<td>Existing moisture content, (% of oven-dry weight)</td>
<td>0.254</td>
</tr>
<tr>
<td>Bulk density, (kg/m3)</td>
<td>1543</td>
</tr>
</tbody>
</table>

Grading of coarse aggregates

To determine the grading and fineness modulus (F.M) of the aggregates, sieve analysis was performed in the laboratory. The experimental results are given below.
Table 4.2: Sieve analysis of coarse aggregates

<table>
<thead>
<tr>
<th>Sieve size</th>
<th>% Weight retain</th>
<th>Cumulative % weight retain</th>
<th>% Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>14 mm</td>
<td>10 mm</td>
<td>7 mm</td>
</tr>
<tr>
<td>19.0 mm (3/4&quot;)</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>9.50 mm (3/8&quot;)</td>
<td>74.17</td>
<td>4.19</td>
<td>0.00</td>
</tr>
<tr>
<td>4.75 mm (#4)</td>
<td>20.82</td>
<td>91.34</td>
<td>75.99</td>
</tr>
<tr>
<td>2.36 mm (#8)</td>
<td>3.02</td>
<td>3.53</td>
<td>21.83</td>
</tr>
<tr>
<td>1.18 mm (#16)</td>
<td>0.84</td>
<td>0.52</td>
<td>0.76</td>
</tr>
<tr>
<td>600 µm (#30)</td>
<td>0.24</td>
<td>0.11</td>
<td>0.17</td>
</tr>
<tr>
<td>300 µm (#50)</td>
<td>0.18</td>
<td>0.06</td>
<td>0.10</td>
</tr>
<tr>
<td>150 µm (#100)</td>
<td>0.23</td>
<td>0.06</td>
<td>0.16</td>
</tr>
<tr>
<td>75 µm (#200)</td>
<td>0.27</td>
<td>0.12</td>
<td>0.31</td>
</tr>
<tr>
<td>Pan</td>
<td>0.23</td>
<td>0.07</td>
<td>0.68</td>
</tr>
</tbody>
</table>

Fig. 4.2: Grading curves of coarse aggregates

MW Ferdous  Static Flexural Behaviour of Fly Ash-Based Geopolymer Composite Beam: An alternative railway sleeper

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Fineness modulus (F.M) of coarse aggregates

The F.M is the empirical figure obtained by adding the cumulative percentages retained in the specified series (ASTM Nos. 100, 50, 30, 16, 8, 4) up to the largest sieves used and dividing the sum by 100. The cumulative percentage retained on the 75 µm sieve would not be included in this calculation because it is not in the F.M series. This parameter gives an indication of the probable behaviour of a concrete mix. Although the F.M is generally computed for a fine aggregate, that of a coarse aggregate is needed for some proportioning methods.

The F.M of coarse aggregate = \[ \left\{ \sum (\text{cumulative} \ % \text{ retained on } 19 \ mm, 9.5 \ mm, 4.75 \ mm, 2.36 \ mm, 1.18 \ mm, 600 \ \mu \text{m}, 300 \ \mu \text{m and } 150 \ \mu \text{m sieves}) \right\} / 100 \]

Table 4.3: Fineness modulus of coarse aggregates

<table>
<thead>
<tr>
<th>Sieve size</th>
<th>14 mm</th>
<th>10 mm</th>
<th>7 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fineness modulus</td>
<td>6.64</td>
<td>5.98</td>
<td>5.69</td>
</tr>
</tbody>
</table>

4.4.1.2 Fine aggregate

A fine aggregate in uncrushed form were used in this experiment. The gravimetric procedure from ASTM C128 was followed to determine the specific gravity and absorption capacities of the sand shown in Table 4.4.

Table 4.4: Properties of fine aggregate

<table>
<thead>
<tr>
<th>Properties of aggregate</th>
<th>Measured value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, (OD)</td>
<td>2.57</td>
</tr>
<tr>
<td>Specific gravity, (SSD)</td>
<td>2.60</td>
</tr>
<tr>
<td>Apparent specific gravity</td>
<td>2.65</td>
</tr>
<tr>
<td>Absorption capacity, (% of oven dry weight)</td>
<td>1.174</td>
</tr>
<tr>
<td>Existing moisture content, (% of oven dry weight)</td>
<td>0.202</td>
</tr>
</tbody>
</table>
Grading of fine aggregates

Sieving of the fine aggregates was performed in the laboratory using an 8-inch diameter sieve. The weight retained on each sieve was measured very carefully. Satisfactory grading was obtained from the fine aggregates when according to the requirements of ASTM C33.

Table 4.5: Sieve analysis of fine aggregates

<table>
<thead>
<tr>
<th>Sieve size</th>
<th>% Weight retain</th>
<th>Cumulative % weight retain</th>
<th>% Passing</th>
<th>ASTM C33 % passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.50 mm, (3/8”)</td>
<td>0.00</td>
<td>0.00</td>
<td>100.00</td>
<td>100</td>
</tr>
<tr>
<td>4.75 mm, (#4)</td>
<td>0.61</td>
<td>0.61</td>
<td>99.39</td>
<td>95 to 100</td>
</tr>
<tr>
<td>2.36 mm, (#8)</td>
<td>4.39</td>
<td>5.00</td>
<td>95.00</td>
<td>80 to 100</td>
</tr>
<tr>
<td>1.18 mm, (#16)</td>
<td>13.57</td>
<td>18.57</td>
<td>81.43</td>
<td>50 to 85</td>
</tr>
<tr>
<td>600 µm, (#30)</td>
<td>24.68</td>
<td>43.25</td>
<td>56.75</td>
<td>25 to 60</td>
</tr>
<tr>
<td>300 µm, (#50)</td>
<td>21.28</td>
<td>64.53</td>
<td>35.47</td>
<td>5 to 30</td>
</tr>
<tr>
<td>150 µm, (#100)</td>
<td>20.52</td>
<td>85.05</td>
<td>14.95</td>
<td>0 to 10</td>
</tr>
<tr>
<td>75 µm, (#200)</td>
<td>13.55</td>
<td>98.60</td>
<td>1.40</td>
<td>0 to 3</td>
</tr>
<tr>
<td>Pan</td>
<td>1.40</td>
<td>100.00</td>
<td>0.00</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Fig. 4.3: Grading curve of fine aggregate
Fineness modulus (F.M) of fine aggregates

The F.M of sand was calculated by the following equation-

The F.M of sand = \[
\left(\frac{\text{cumulative % retained on 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm, 600 \mu m, 300 \mu m and 150 \mu m sieves}}{100}\right)
\]
and, thus equals to 2.17.

The fineness modulus of fine aggregates should lie between 2.3 and 3.1 as specified in ASTM C33. The result of 2.17 indicated, finer sand was used to prepare the concrete which requires more liquid for good workability.

Grading of combined aggregates

Before placing the aggregates in the mixing machine, the different sizes of the coarse and fine aggregates were combined to obtain a suitable gradation. For the geopolymer concrete mix, percentage masses of 14 mm, 10mm, 7mm and fine aggregates of 15%, 35%, 20% and 30%, respectively were chosen which satisfied the nearest nominal size grading requirements for All-in aggregate stated in BS 882: 1973.

Table 4.6: Grading of combined aggregates

<table>
<thead>
<tr>
<th>Sieve size</th>
<th>% Passing of aggregates</th>
<th>Combination</th>
<th>20 mm nominal size BS 882: 1973</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm, (3/4&quot;)</td>
<td>100.00</td>
<td>100.00</td>
<td>100.00</td>
</tr>
<tr>
<td>9.50 mm, (3/8&quot;)</td>
<td>25.83</td>
<td>95.81</td>
<td>100.00</td>
</tr>
<tr>
<td>4.75 mm, (#4)</td>
<td>5.01</td>
<td>4.47</td>
<td>24.01</td>
</tr>
<tr>
<td>2.36 mm, (#8)</td>
<td>1.99</td>
<td>0.94</td>
<td>2.18</td>
</tr>
<tr>
<td>1.18 mm, (#16)</td>
<td>1.15</td>
<td>0.42</td>
<td>1.42</td>
</tr>
<tr>
<td>600 \mu m, (#30)</td>
<td>0.91</td>
<td>0.31</td>
<td>1.25</td>
</tr>
<tr>
<td>300 \mu m, (#50)</td>
<td>0.73</td>
<td>0.25</td>
<td>1.15</td>
</tr>
<tr>
<td>150 \mu m, (#100)</td>
<td>0.50</td>
<td>0.19</td>
<td>0.99</td>
</tr>
<tr>
<td>75 \mu m (#200)</td>
<td>0.23</td>
<td>0.07</td>
<td>0.68</td>
</tr>
</tbody>
</table>

MW Ferdous
Static Flexural Behaviour of Fly Ash-Based Geopolymer Composite Beam: An alternative railway sleeper
4.4.2 Fly ash

Fly ash was obtained from the Boral Company, Australia, and used in this research as the main constituent of the binding materials in geopolymer concrete. Its specific gravity was measured using the same procedure described in ASTM C188 for cement. The result of 2.06 indicated that the density of the fly ash was much lower than that of traditional cement. The XRF analysis showed that the percentage sum of SiO$_2$, Al$_2$O$_3$ and Fe$_2$O$_3$ in the fly ash was around 93% which ensured that the fly ash used was a Class F type.

Table 4.7: Chemical composition of fly ash

<table>
<thead>
<tr>
<th>Oxide</th>
<th>SiO$_2$ (%)</th>
<th>CaO (%)</th>
<th>Al$_2$O$_3$ (%)</th>
<th>MgO (%)</th>
<th>Fe$_2$O$_3$ (%)</th>
<th>SO$_3$ (%)</th>
<th>TiO$_2$ (%)</th>
<th>Na$_2$O (%)</th>
<th>K$_2$O (%)</th>
<th>L.O.I</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>62.19</td>
<td>1.97</td>
<td>27.15</td>
<td>0.40</td>
<td>3.23</td>
<td>0.07</td>
<td>1.06</td>
<td>0.30</td>
<td>0.89</td>
<td>1.75</td>
</tr>
</tbody>
</table>
4.4.3 Alkaline liquid

In the present experimental work, a combination of sodium silicate (Na$_2$SiO$_3$) and sodium hydroxide (NaOH) solutions was chosen as the alkaline liquid. Sometimes, a potassium-based solution is used to prepare geopolymer concrete but it was not used here due to its higher cost.

4.4.4 Sodium silicate solution

The sodium silicate solution was obtained from IMCD Australia Limited. This solution is recommended for use as a detergent ingredient, adhesive, binder, feedstock silica source or industrial raw material. Some of its important properties of the solution are tabulated in Table 4.8.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composition</td>
<td></td>
</tr>
<tr>
<td>Sodium oxide, (Na$_2$O)</td>
<td>14.7 (%)</td>
</tr>
<tr>
<td>Silicon dioxide, (SiO$_2$)</td>
<td>29.4 (%)</td>
</tr>
<tr>
<td>Water</td>
<td>55.9 (%)</td>
</tr>
<tr>
<td>Specific gravity of solution</td>
<td>1.52</td>
</tr>
<tr>
<td>pH value</td>
<td>11 to 13</td>
</tr>
<tr>
<td>Odour</td>
<td>No odour</td>
</tr>
<tr>
<td>Solubility in water</td>
<td>Completely soluble</td>
</tr>
</tbody>
</table>

4.4.5 Sodium hydroxide solution

The sodium hydroxide solution was prepared in the laboratory by dissolving sodium hydroxide pellets in water. Its specific gravity depended on its concentration expressed by the term molar (M). Generally, the concentration for making geopolymer concrete varies from 8M to 16M. In 2005, Hardjito and Rangan measured the mass of a NaOH solid in different concentrations of this solution. According to their measurements, an
8M solution contains 262 grams of NaOH solid per kg of solution whereas 10M, 12M, 14M and 16M solutions have 314 grams, 361 grams, 404 grams and 444 grams, respectively. From this information, the specific gravity of a NaOH solution can be calculated; for example, the actual weight of a 1 litre 16M solution is 
\[(16 \times 40 \times 1000)/444 = 1441\text{ grams},\] 
where 40 is the molecular weight of the NaOH, and its specific gravity is 
\[1441/1000 = 1.44.\]

In this research, a 16M solution was chosen to achieve a higher strength concrete. Its properties are given in Table 4.9.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composition, (16 molar solution)</td>
<td></td>
</tr>
<tr>
<td>Sodium hydroxide (NaOH) solid</td>
<td>44.4 (%)</td>
</tr>
<tr>
<td>Water</td>
<td>55.6 (%)</td>
</tr>
<tr>
<td>Specific gravity, (16 molar solution)</td>
<td>1.44</td>
</tr>
</tbody>
</table>

### 4.4.6 Super-plasticiser

A super-plasticiser was used to improve the workability of fresh geopolymer concrete. A carboxylic ether polymer-based super-plasticiser under the brand name ADVA 142 was applied in the concrete mix. Its addition rates can vary between 400 and 1200 ml per 100 kg of total cementitious materials depending on its application. The specific gravity of the super-plasticiser was 1.082.

### 4.5 Mixing, Casting and Curing of Geopolymer Concrete

#### 4.5.1 Mixing

All mixings for cylinders were performed manually in the laboratory using a pan mixer and the procedure was as follows.
The aggregates were first mixed in the pan mixer and then the water required for their absorption was added. The mixing time was kept to two minutes so that the aggregates would acquire as near as possible, a saturated surface dry conditions.

Fly ash was then added to the aggregates and mixed for around two minutes.

Alkaline solutions were prepared by mixing the sodium hydroxide and sodium silicate solutions at least one day before concrete mixing and were gradually added to the solids and mixed for another two minutes.

Finally, the super-plasticiser was added to the plastic mix and the mixing was continued, usually for two minutes, until the binding paste covered all the aggregates and the colour of the mixture was uniformly dark.

Eleven sets of cylinders were prepared in the laboratory. The amounts of ingredients in every mix were obtained following the design method proposed in this chapter which will be detailed later. In the first trial, mixes 1 to 5 were prepared taking a constant amount of fly ash of 320 kg/m³, with an increased alkaline liquid to fly ash ratio of 0.76, 0.80, 0.85, 0.90 and 0.95. It was observed, the compressive strength decreased gradually with the increases of alkaline liquid to fly ash ratio. Consequently, in the next trial, mixes 6 to 9 were prepared where a constant alkaline liquid to fly ash ratio of 0.76 were used in all mixes. This time, an increased amount of fly ash of 340, 360, 380 and 400 kg/m³ were used in mixes 6 to 9 respectively. The strength values obtained for mixes 6 to 9 were more or less similar and that may be due to using the same water-to-geopolymer solids ratio. In the third trial, mixes 10 and 11 were cast keeping a constant amount of fly ash of 400 kg/m³. But this time, the alkaline liquid to fly ash ratio was
chosen as 0.65 and 0.55 respectively with the aim of achieving higher strength of concrete.

Table 4.10: Amounts of ingredients in mix

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Aggregates in SSD, (kg/m³)</th>
<th>Fly ash, (kg/m³)</th>
<th>Na₂SiO₃ Solution, (kg/m³)</th>
<th>NaOH Solution, (kg/m³)</th>
<th>Superplasticiser, (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>14 mm</td>
<td>10 mm</td>
<td>7 mm</td>
<td>Fine</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>253</td>
<td>591</td>
<td>340</td>
<td>509</td>
<td>319</td>
</tr>
<tr>
<td>2</td>
<td>250</td>
<td>584</td>
<td>336</td>
<td>502</td>
<td>319</td>
</tr>
<tr>
<td>3</td>
<td>246</td>
<td>574</td>
<td>330</td>
<td>494</td>
<td>319</td>
</tr>
<tr>
<td>4</td>
<td>241</td>
<td>564</td>
<td>324</td>
<td>485</td>
<td>319</td>
</tr>
<tr>
<td>5</td>
<td>237</td>
<td>554</td>
<td>319</td>
<td>477</td>
<td>319</td>
</tr>
<tr>
<td>6</td>
<td>245</td>
<td>573</td>
<td>329</td>
<td>493</td>
<td>339</td>
</tr>
<tr>
<td>7</td>
<td>237</td>
<td>554</td>
<td>319</td>
<td>477</td>
<td>359</td>
</tr>
<tr>
<td>8</td>
<td>229</td>
<td>536</td>
<td>308</td>
<td>461</td>
<td>379</td>
</tr>
<tr>
<td>9</td>
<td>221</td>
<td>517</td>
<td>297</td>
<td>445</td>
<td>399</td>
</tr>
<tr>
<td>10</td>
<td>233</td>
<td>544</td>
<td>313</td>
<td>468</td>
<td>399</td>
</tr>
<tr>
<td>11</td>
<td>244</td>
<td>569</td>
<td>327</td>
<td>489</td>
<td>399</td>
</tr>
</tbody>
</table>

4.5.2 Casting

The fresh concrete was cast and compacted by the method normally used for portland cement concrete. In the plastic state of the mix, the fly ash-based geopolymer concrete was cohesive and dark in colour. The cylinders were cast in 75×150 mm plastic moulds because the geopolymer concrete has shown strong adhesion to steel moulds which would have made de-moulding after hardening very difficult. A vibration table was used to compact the specimens, as shown in Fig. 4.5.
4.5.3 Curing

The process of geopolymerisation requires curing at an elevated temperature. A longer curing time can improve the polymerisation process and give higher compressive strength. The specimens were kept at rest for 24 hours after casting until they were placed in oven. However as, after this time, they were not sufficiently stiff to be demoulded, it was decided to cure them in their plastic moulds in the oven at 60°C for 3 days. At the end of this curing period, the moulds were taken out of the oven and each cylinder retrieved by splitting the surface of its plastic mould using a sharp knife after it had cooled to room temperature. The specimens were then placed in a temperature-controlled room (room temperature 23°C and humidity 50%) prior to testing.
4.6 Grinding of Cylinders

The cylindrical specimens were surface-ground before being placed in the testing machine mainly to smooth their cross-sectional surface areas to avoid the need for any capping. Once finished, a highly smooth surface of each cylinder was obtained which represented high accuracy in terms of load distribution, shape and dimensions.
4.7 Compressive Strength Testing of Geopolymer Concrete

The compressive strength of concrete is the most common performance measure criterion in the design of concrete structures. It is actually the measure of the concrete’s ability to resist a load which might crush it. To measure the compressive strength, three 75×150 mm size cylinders were tested in every mix according to AS 1012.9 (1999) using a 3000 kN capacity testing machine. The geopolymer concrete showed very consistent results in every mix.

4.8 Water-to-geopolymer Solids Ratio of Mixture

Like, the water-cement ratio and compressive strength of OPC concrete, geopolymer concrete has an inverse relationship between the water-to-geopolymer solids ratio and compressive strength, as shown in Fig. 4.8.

![Graph showing variations in compressive strength with water-to-geopolymer solids ratio](image)

**Fig. 4.8:** Variations in compressive strength with water-to-geopolymer solids ratio
In geopolymer concrete, the total mass of water is the sum of the masses of water contained in the sodium silicate and sodium hydroxide solutions. On the other hand, geopolymer solids are calculated as the sum of the masses of the fly ash, and sodium silicate and sodium hydroxide solids. It has been observed that the water-to-geopolymer solids ratio linearly increases with increases in the alkaline liquid-to-fly ash ratio if the molarity of the NaOH solution and the ratio of the sodium silicate-to-sodium hydroxide solutions remain the same in all mixes. Variations in the water-to-geopolymer solids ratio with the alkaline liquid-to-fly ash ratio are depicted in Fig. 4.9. This information is useful at the start of mix design when the water-to-geopolymer solids ratio has still not been clearly determined by the designer.

![Fig. 4.9: Water-to-geopolymer solids ratio vs alkaline liquid-to-fly ash ratio](image-url)
4.9 Design Graph for Geopolymer Concrete

A design graph with respect to the two major variables that have a significant effect on the water-to-geopolymer solids ratio was prepared. These two variables are the alkaline liquid-to-fly ash ratio and the amount of fly ash in the mixture. The addition of extra water to improve the workability of the mix had an influence on the alkaline liquid-to-fly ash ratio obtained at the end of the mix design which was different from the initial ratio and is called the true ratio. Variations in compressive strength with the true ratio of the alkaline liquid-to-fly ash for different fly ash contents are plotted in Fig. 4.10.

![Fig. 4.10: Strength vs alkaline liquid-to-fly ash ratio with different ash contents](image)

The other variables that can influence the compressive strength of geopolymer concrete were kept constant at 16M for the molarity of the NaOH solution, at 2.3 for the sodium silicate solution-to-sodium hydroxide solution ratio and at 3 days curing at 60°C in the
oven. In addition, a super-plasticiser in an amount of 1% of the mass of the fly ash was used in this mix. Moreover, three different sizes of coarse aggregates (14 mm, 10 mm and 7 mm) were used with fine aggregates in a ratio of 70/30 by mass. The average specific gravity of aggregates was 2.6 and the percentage mass of the Na₂O solid, SiO₂ solid and water in the sodium silicate solution were 14.7, 29.4 and 55.9, respectively. Some data in Fig. 4.10 were obtained from laboratory experiments and the others calculated by interpolating the experimental values.

It can be seen in Fig. 4.10 that the optimum compressive strength was obtained when the volume of the fly ash content was 340 kg/m³ compared with other volumes with the same alkaline liquid-to-fly ash ratio. This indicates that more fly ash content in the mix did not lead to the concrete having more strength. The strength of geopolymer concrete depends mainly on its water-to-geopolymer solids ratio, as discussed earlier and shown in Fig. 4.8.

**4.10 Relationship between 28-day and 7-day Compressive Strengths**

Sometimes, in a large project, concreting has to be done in several stages. It is very time consuming to wait 28 days when different concrete strengths needed at different stages and increases the duration of project. The proposed technique may be helpful for predicting the 28-day strength of fly ash-based geopolymer concrete during a 7-day period, thereby significantly reducing the duration of project execution.
Fig. 4.11 compares the relationships between 28-day and 7-day compressive strengths of ordinary portland cement (OPC) concrete and geopolymer concrete. The data required to plot the relationship for geopolymer concrete were obtained from laboratory experiments in which the geopolymer cylinders were heated at 60°C for 3 days in the oven. A range of mixes were also made in the laboratory for OPC concrete to select the suitable mix for normal concrete composite beam. The laboratory test data and the following simple empirical relationship given by Davis et al. in 1964 [115] which is still being used to predict the normal concrete compressive strength [45] were used to plot the curve for OPC concrete.

\[
f_{c(28)}' = f_{c(7)}' + \frac{30}{\sqrt{145}} \sqrt{f_{c(7)}}
\]  \hspace{1cm} (4.1)

\(f_{c(7)}\) and \(f_{c(28)}\) are the 7-day and 28-day compressive strengths, respectively, in MPa.
The most valuable aspect noted in Fig. 4.11 is that the compressive strength of OPC concrete at 28 days is around 1.5 times that at 7 days whereas, for geopolymer concrete this relationship is only 1.15 times on average. This indicates that geopolymer concrete gained strength rapidly at an earlier stage than OPC concrete. In other words, geopolymer concrete achieved 87 percent of its 28-day strength in 7 days while OPC concrete achieved only 67 percent.

4.11 Proposed Mix Design Procedure with Example

The limitations of recently published design methods reported by other workers [59] in this field motivated the establishment of the rigorous method of design described step by step below. This procedure uses a typical example for illustration purposes which, along with others, is worked in a spreadsheet EXCEL routine. Screen prints of the procedure as it appears in the spreadsheet are also presented. By adopting this program, the user should have no difficulty choosing different proportions or adapting to different conditions.
Let’s target a mean compressive strength at 28 days for geopolymer is 45 MPa,

Given the data in Table 4.11

**Table 4.11: Material properties of concrete ingredients**

<table>
<thead>
<tr>
<th>Materials</th>
<th>Specific gravity</th>
<th>F.M</th>
<th>Absorption capacity, (% of OD)</th>
<th>Moisture content, (% of OD)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 mm aggregate</td>
<td>2.65 (OD)</td>
<td>6.64</td>
<td>0.675</td>
<td>0.254</td>
<td>-</td>
</tr>
<tr>
<td>10 mm aggregate</td>
<td>2.63 (OD)</td>
<td>5.98</td>
<td>0.772</td>
<td>0.316</td>
<td>-</td>
</tr>
<tr>
<td>7 mm aggregate</td>
<td>2.59 (OD)</td>
<td>5.69</td>
<td>1.382</td>
<td>0.445</td>
<td>-</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>2.57 (OD)</td>
<td>2.17</td>
<td>1.174</td>
<td>0.202</td>
<td>-</td>
</tr>
<tr>
<td>Fly ash</td>
<td>2.06</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Na$_2$SiO$_3$ solution</td>
<td>1.52</td>
<td>Na$_2$O =14.7%, SiO$_2$ = 29.4%, H$_2$O=55.9%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Superplasticiser</td>
<td>1.082</td>
<td>400-1200ml/100 kgs of cementitious materials</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Entrapped air</td>
<td></td>
<td>Average value after several trial mixes = 3.29 %</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

**Step 1: Requirements for weight of fly ash and alkaline liquid**

In this design, the first main factor is the amount of fly ash content. As the most costly ingredient in a geopolymer concrete mix is alkaline liquid, for an economic design of concrete, designers should try to use minimum amounts of it in their mixes. Fig. 4.10 shows that, for a 45 MPa concrete, the alkaline liquid requirements are lower when the fly ash content is 320 kg/m$^3$ and the corresponding alkaline liquid-to-fly ash ratio is 0.76. To increase the workability of the mixture (based on previous trials), a superplasticiser and water, each in the amount of 1% of the total amount of fly ash weight, are added. Although the alkaline liquid-to-fly ash ratio does not depend on the addition of a super-plasticiser, it can increase with the addition of water for workability. It can be observed that, the addition of X% of water increases the alkaline liquid-to-fly ash ratios by roughly 0.02X. Therefore, the design starts by taking an alkaline liquid-to-fly ash
ratio = 0.76-0.02×1= 0.74 which could be increased to 0.76 and is termed the true ratio at the end of the mix design.

Therefore, if the amount of fly ash = 320 kg/m³

the alkaline liquid required = 320 × 0.74 = 237 kg/m³.

**Step 2: Addition of chemical admixture or water for workability (if needed)**

Taking the dosage of a super-plasticiser as 1% of the fly ash weight,

and its amount = 320 × (1/100) = 3.2 kg/m³.

The specific gravity of the super-plasticiser = 1.082 and the approximate rate of addition of a super-plasticiser = (3.2×100×1000)/(320×1.082) = 924 ml/100kg fly ash which is in the range 400 to 1200 ml/100kg recommended by the manufacturer.

Taking the addition of extra water as 1% of the fly ash weight

the amount of extra water = 320 × (1/100) = 3.2 kg/m³.

**Step 3: Calculation and preparation of alkaline liquids**

Taking, the Na₂SiO₃ solution-to-NaOH solution ratio (from experience [59]) = 2.5

the mass of the Na₂SiO₃ solution + NaOH solution = 237 kg/m³

Therefore, the NaOH solution required = 237/ (1+2.5) = 68 kg/m³

and Na₂SiO₃ solution required = 237-68 = 169 kg/m³

**Preparation of NaOH solution**

Taking the molarity of the NaOH solution = 16 Mole,

as discussed earlier, this can be prepared by mixing 44.4% of a NaOH solid with 55.6% of water.

Then, the specific gravity of the 16M NaOH solution = [(100/44.4) × (16×40)]/1000 = 1.44,

where 40 is the molecular weight of the NaOH solid.
Therefore, the NaOH solid required = 68× (44.4/100) = 30 kg/m³ and 
the water required = 68× (55.6/100) = 38 kg/m³.

In this design, as 3.2 kg of extra water is required to improve workability 
the total amount of water in solution = 38+3.2 = 41 kg/m³.

To keep the concentration (16M) of the NaOH solution constant, the NaOH solid is now 
recalculated as 
NaOH solid required = 41× (44.4/55.6) = 33 kg/m³.

**Summarised form of alkaline liquids**

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Na₂SiO₃ solution required</td>
<td>169</td>
<td>kg/m³ of concrete</td>
</tr>
<tr>
<td>NaOH solution required</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NaOH solid</td>
<td>33 kg</td>
<td>kg/m³ of concrete</td>
</tr>
<tr>
<td>Water</td>
<td>41 kg</td>
<td>kg/m³ of concrete</td>
</tr>
<tr>
<td>Molarity of NaOH solution</td>
<td>16 M</td>
<td>Mole</td>
</tr>
</tbody>
</table>

**Step 4: Required weight/volume of coarse aggregate and fine aggregates**

The volume occupied by fly ash = 320/ (2.06×1000) = 0.1553 m³.

The volume occupied by the NaOH solution = 74/ (1.44×1000) = 0.0510 m³.

The volume occupied by the Na₂SiO₃ solution = 169/ (1.52×1000) = 0.1113 m³.

The volume occupied by entrapped air = 3.29/100 = 0.0329 m³.

The total volume occupied by these constituents = 0.3505 m³.

To fulfil the grading requirements of the aggregates, it is necessary to mix the 14 mm, 
10mm, 7mm and fine aggregates by 15%, 35%, 20% and 30%, respectively.

The combined F.M = (15×6.64+35×5.98+20×5.69+30×2.17)/100 = 4.88.

The combined Sp.G of coarse aggregate = (15×2.65+35×2.63+20×2.59)/70 = 2.62

The volume of the coarse and fine aggregates = 1-0.3505 = 0.6495 m³.
The oven-dry (OD) masses of the coarse and fine aggregates are 70% and 30%, respectively.

The volume factor of the coarse aggregates = 70/ (2.62×1000) = 0.02669.

The volume factor of the fine aggregate = 30/ (2.57×1000) = 0.01167.

Therefore, the OD volume of the coarse aggregates

\[ \frac{0.02669}{(0.02669+0.01167)} \times 100 = 69.57\% \]

and that of the fine agg. = \[ \frac{0.01167}{(0.02669+0.01167)} \times 100 = 30.43\% \]

Now, the actual volume of coarse aggregates = 0.6495× (69.57/100) = 0.4519 m³

and that of the fine aggregate = 0.6495× (30.43/100) = 0.1976 m³

Therefore, the total weight of coarse aggregate = 0.4519×2.62×1000 = 1185 kg/m³

and that of the fine aggregate = 0.1976×2.57×1000 = 508 kg/m³

**Summarised form of oven-dry aggregates**

<table>
<thead>
<tr>
<th>Type of aggregates</th>
<th>Calculations</th>
<th>Amount (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14 mm (15%)</td>
<td>1185×15/(15+35+20)</td>
<td>254</td>
</tr>
<tr>
<td>10 mm (35%)</td>
<td>1185×35/(15+35+20)</td>
<td>593</td>
</tr>
<tr>
<td>7 mm (20%)</td>
<td>1185×20/(15+35+20)</td>
<td>339</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>Sand (30%)</td>
<td>508</td>
</tr>
</tbody>
</table>

**Step 5: Adjustment of absorption capacities and moisture contents of aggregates**

<table>
<thead>
<tr>
<th>Size</th>
<th>Weight of oven-dry aggregate</th>
<th>Absorption capacity, (% of oven-dry weight)</th>
<th>Moisture content, (% of oven-dry weight)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 mm</td>
<td>254</td>
<td>0.675</td>
<td>0.254</td>
</tr>
<tr>
<td>10 mm</td>
<td>593</td>
<td>0.772</td>
<td>0.316</td>
</tr>
<tr>
<td>7 mm</td>
<td>339</td>
<td>1.382</td>
<td>0.445</td>
</tr>
<tr>
<td>Fine</td>
<td>508</td>
<td>1.174</td>
<td>0.202</td>
</tr>
</tbody>
</table>

Therefore, the amount of water required for the absorption of the oven-dry aggregates

\[ (254×0.675+593×0.772+339×1.382+508×1.174)/100 = 17 \text{ kg/m}^3 \text{ of concrete.} \]
### Step 6: Calculation of total volume

<table>
<thead>
<tr>
<th>Ingredients</th>
<th>Amount, (kg/m³)</th>
<th>Sp.G</th>
<th>Volume, (m³)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregates</td>
<td>1185</td>
<td>2.62</td>
<td>0.4519</td>
<td>Oven-dry aggregates</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>508</td>
<td>2.57</td>
<td>0.1976</td>
<td>Oven-dry aggregate</td>
</tr>
<tr>
<td>Fly ash</td>
<td>320</td>
<td>2.06</td>
<td>0.1553</td>
<td></td>
</tr>
<tr>
<td>Na₂SiO₃ solution</td>
<td>169</td>
<td>1.52</td>
<td>0.1113</td>
<td></td>
</tr>
<tr>
<td>NaOH solution</td>
<td>74</td>
<td>1.44</td>
<td>0.0510</td>
<td></td>
</tr>
<tr>
<td>Super-plasticiser</td>
<td>3.2</td>
<td>1.082</td>
<td>0.0030</td>
<td></td>
</tr>
<tr>
<td>Entrapped air (%)</td>
<td>3.29</td>
<td>-</td>
<td>0.0329</td>
<td></td>
</tr>
<tr>
<td>Absorption water</td>
<td>17</td>
<td>1</td>
<td>0.0000</td>
<td>Volume already accounted</td>
</tr>
</tbody>
</table>

As the total resultant volume 0.3% more than 1 m³ due to the use of a super-plasticiser, we need to adjust the volume, as discussed below.

### Step 7: Volume adjustment (only when using super-plasticiser)

The volume could be adjusted by dividing the amount of each ingredient by 1.0030, as shown below.

<table>
<thead>
<tr>
<th>Ingredients</th>
<th>Amount, (kg/m³)</th>
<th>Sp.G</th>
<th>Volume, (m³)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregates</td>
<td>1182</td>
<td>2.62</td>
<td>0.4505</td>
<td>Oven-dry aggregates</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>506</td>
<td>2.57</td>
<td>0.1971</td>
<td>Oven-dry aggregate</td>
</tr>
<tr>
<td>Fly ash</td>
<td>319</td>
<td>2.06</td>
<td>0.1549</td>
<td></td>
</tr>
<tr>
<td>Na₂SiO₃ solution</td>
<td>169</td>
<td>1.52</td>
<td>0.1110</td>
<td></td>
</tr>
<tr>
<td>NaOH solution</td>
<td>73</td>
<td>1.44</td>
<td>0.0508</td>
<td></td>
</tr>
<tr>
<td>Super-plasticiser</td>
<td>3.19</td>
<td>1.082</td>
<td>0.0029</td>
<td></td>
</tr>
<tr>
<td>Entrapped air (%)</td>
<td>3.28</td>
<td>-</td>
<td>0.0328</td>
<td></td>
</tr>
<tr>
<td>Absorption water</td>
<td>17</td>
<td>1</td>
<td>0.0000</td>
<td>Volume already accounted</td>
</tr>
</tbody>
</table>

Total volume = 1.0000 Total volume = 1 m³
### Step 8: Details of mixing for oven-dry aggregates (1 m³ of concrete)

The required amount of a 14 mm coarse aggregate = 1182×15/ (15+35+20) = 253 kg. Similarly, those of the 10 mm and 7 mm aggregates are 591 kg and 338 kg, respectively, as shown below.

<table>
<thead>
<tr>
<th>Ingredients</th>
<th>Amount, (kg/m³)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14 mm</td>
<td>253</td>
<td>Oven-dry aggregate</td>
</tr>
<tr>
<td>10 mm</td>
<td>591</td>
<td>Oven-dry aggregate</td>
</tr>
<tr>
<td>7 mm</td>
<td>338</td>
<td>Oven-dry aggregate</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>506</td>
<td>Oven-dry aggregate</td>
</tr>
<tr>
<td>Fly ash</td>
<td>319</td>
<td></td>
</tr>
<tr>
<td>Na₂SiO₃ solution</td>
<td>169</td>
<td></td>
</tr>
<tr>
<td>NaOH solution</td>
<td>73</td>
<td>16 Molar</td>
</tr>
<tr>
<td>Superplasticiser</td>
<td>3.19</td>
<td></td>
</tr>
<tr>
<td>Water required for absorption</td>
<td>17</td>
<td></td>
</tr>
</tbody>
</table>

As it is difficult to obtain oven-dry aggregates at the time of mixing, a better procedure is to prepare the chart of mix details considering aggregates as they are found in field conditions.

### Step 9: Details of mixing for aggregates in field conditions (1 m³ of concrete)

Considering the moisture content in this example, the required amount for the 14 mm coarse aggregate = 253× (1+0.254/100) = 254 kg and, for the 10 mm, 7 mm and fine aggregates 593 kg, 339 kg and 507 kg, respectively.

Therefore, for the field conditions in this example, the amount of water required for the absorption of aggregates

\[= 17-(254-253)-(593-591)-(339-338)-(507-506)\]

\[= 12 \text{ kg.}\]
### Ingredients

<table>
<thead>
<tr>
<th>Ingredients</th>
<th>Amount, (kg/m³)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14 mm</td>
<td>254</td>
<td>Aggregate in field condition</td>
</tr>
<tr>
<td>10 mm</td>
<td>593</td>
<td>Aggregate in field condition</td>
</tr>
<tr>
<td>7 mm</td>
<td>339</td>
<td>Aggregate in field condition</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>507</td>
<td>Aggregate in field condition</td>
</tr>
<tr>
<td>Fly ash</td>
<td>319</td>
<td></td>
</tr>
<tr>
<td>Na₂SiO₃ solution</td>
<td>169</td>
<td></td>
</tr>
<tr>
<td>NaOH solution</td>
<td>73</td>
<td>16 Molar</td>
</tr>
<tr>
<td>Super-plasticiser</td>
<td>3.19</td>
<td></td>
</tr>
<tr>
<td>Water required for absorption</td>
<td>12</td>
<td></td>
</tr>
</tbody>
</table>

The true ratio of alkaline liquid-to-fly ash = (169+73)/319 = 0.76 (as mentioned earlier) and that of the Na₂SiO₃-to-NaOH solutions = 169/73 = 2.3.

The density of concrete,
\[= \frac{254+593+339+507+319+169+73+3.19+12}{2269} = 2269 \text{ kg/m}^3.\]

Fly ash: Fine aggregate: Coarse aggregate = 1: 1.59: 3.71

These relative ratios are calculated considering the aggregates in oven dry condition.

#### Step 10: Water-to-geopolymer solids ratio calculation

<table>
<thead>
<tr>
<th>Type of solution</th>
<th>Amount, (kg)</th>
<th>Ingredients</th>
<th>Percentage</th>
<th>Weight, (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Na₂SiO₃ solution</td>
<td>169</td>
<td>Na₂O solid</td>
<td>14.7</td>
<td>24.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SiO₂ solid</td>
<td>29.4</td>
<td>49.58</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Water</td>
<td>55.9</td>
<td>94.27</td>
</tr>
<tr>
<td>NaOH solution</td>
<td>73</td>
<td>NaOH solid</td>
<td>44.4</td>
<td>32.41</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Water</td>
<td>55.6</td>
<td>40.70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fly ash</td>
<td>-</td>
<td>319</td>
</tr>
</tbody>
</table>

Therefore, the water-to-geopolymer solids ratio
\[= \frac{94.27+40.70}{24.79+49.58+32.41+319} = 0.32.\]
Table 4.12: Details of concrete mixes

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>14 mm</th>
<th>10 mm</th>
<th>7 mm</th>
<th>Fine</th>
<th>Fly ash, (kg)</th>
<th>Sodium silicate solution, (kg)</th>
<th>NaOH solution Mass, (kg)</th>
<th>Molarity, (M)</th>
<th>Superplasticiser, (kg)</th>
<th>Rest before curing, (days)</th>
<th>Curing</th>
<th>Workability, (mm)</th>
<th>Density, (kg/m³)</th>
<th>Compressive strength, (MPa)</th>
<th>7 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>253</td>
<td>591</td>
<td>340</td>
<td>509</td>
<td>319</td>
<td>173</td>
<td>75</td>
<td>16</td>
<td>3.2</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>170</td>
<td>2286</td>
<td>34.74</td>
<td>42.67</td>
</tr>
<tr>
<td>2</td>
<td>250</td>
<td>584</td>
<td>336</td>
<td>502</td>
<td>319</td>
<td>182</td>
<td>79</td>
<td>16</td>
<td>3.2</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>175</td>
<td>2281</td>
<td>34.11</td>
<td>40.24</td>
</tr>
<tr>
<td>3</td>
<td>246</td>
<td>574</td>
<td>330</td>
<td>494</td>
<td>319</td>
<td>194</td>
<td>83</td>
<td>16</td>
<td>3.2</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>190</td>
<td>2269</td>
<td>32.08</td>
<td>37.54</td>
</tr>
<tr>
<td>4</td>
<td>241</td>
<td>564</td>
<td>324</td>
<td>485</td>
<td>319</td>
<td>205</td>
<td>88</td>
<td>16</td>
<td>3.2</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>200</td>
<td>2237</td>
<td>29.86</td>
<td>33.75</td>
</tr>
<tr>
<td>5</td>
<td>237</td>
<td>554</td>
<td>319</td>
<td>477</td>
<td>319</td>
<td>217</td>
<td>92</td>
<td>16</td>
<td>3.2</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>210</td>
<td>2234</td>
<td>27.81</td>
<td>30.17</td>
</tr>
<tr>
<td>6</td>
<td>245</td>
<td>573</td>
<td>329</td>
<td>493</td>
<td>339</td>
<td>184</td>
<td>80</td>
<td>16</td>
<td>3.4</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>160</td>
<td>2248</td>
<td>40.55</td>
<td>48.01</td>
</tr>
<tr>
<td>7</td>
<td>237</td>
<td>554</td>
<td>319</td>
<td>477</td>
<td>359</td>
<td>195</td>
<td>84</td>
<td>16</td>
<td>3.6</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>170</td>
<td>2242</td>
<td>40.46</td>
<td>47.35</td>
</tr>
<tr>
<td>8</td>
<td>229</td>
<td>536</td>
<td>308</td>
<td>461</td>
<td>379</td>
<td>206</td>
<td>89</td>
<td>16</td>
<td>3.8</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>175</td>
<td>2219</td>
<td>40.37</td>
<td>46.31</td>
</tr>
<tr>
<td>9</td>
<td>221</td>
<td>517</td>
<td>297</td>
<td>445</td>
<td>399</td>
<td>216</td>
<td>94</td>
<td>16</td>
<td>4.0</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>180</td>
<td>2207</td>
<td>38.40</td>
<td>44.19</td>
</tr>
<tr>
<td>10</td>
<td>233</td>
<td>544</td>
<td>313</td>
<td>468</td>
<td>399</td>
<td>185</td>
<td>81</td>
<td>16</td>
<td>4.0</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>150</td>
<td>2237</td>
<td>49.82</td>
<td>56.67</td>
</tr>
<tr>
<td>11</td>
<td>244</td>
<td>569</td>
<td>327</td>
<td>489</td>
<td>399</td>
<td>157</td>
<td>70</td>
<td>16</td>
<td>4.0</td>
<td>1</td>
<td>3</td>
<td>60</td>
<td>90</td>
<td>2235</td>
<td>61.07</td>
<td>63.46</td>
</tr>
</tbody>
</table>

Shown above are the 11 trial mixes, Mix number 1 to 5 showed higher workability compared to the others whereas mixes number 6 to 9 were more consistent result in compressive strength, density and workability. Although having the difficulties in vibrating of concrete, mixes number 10 and 11 showed the best performances in strength point of view. In general, the best mix design in Table 4.12 is that which fulfils the strength and workability requirements with lower cost. The most costly ingredient in a geopolymer concrete mix is alkaline liquid, for an economic design of concrete, designers should try to use a minimum amount.
4.12 Conclusions

In this chapter, a detailed procedure for the design of a fly ash-based geopolymer concrete mixture has been proposed and illustrated using an example. A range of mixes were made to test the method using various water-to-geopolymer solids ratios and different amounts of fly ash content. It was found that the compressive strength of fly ash-based geopolymer concrete decreased linearly with increases in the water-to-geopolymer solids ratio. This observation is in agreement with the basic principles of ordinary portland cement concrete, the strength of which decreases with increases in the water-cement ratio.

In Chapter 5, the preparation of environmentally friendly composite sleepers and their static performances are presented.
Chapter 5

Investigation into the Static Flexural Behaviour of Composite Sleepers

5.1 General

Pultruded FRP profiles [17, 18] and geopolymer concrete [19-21] have many advantages that could favour their application in railway sleepers. Yet, up to now, there has been no attempt at using these materials for manufacturing sleeper. This chapter investigates the load-deflection behaviour and failure mechanisms of a composite railway sleeper made of a hollow rectangular pultruded profile filled up with geopolymer concrete. The concrete inside the profile provides internal support, preventing local buckling and, contributes to the inertia of the section and internal forces, which can increases the flexural strength of the beam. A comparative study of the flexural behaviours of existing railway sleepers such as those made of timber, prestressed concrete and steel, and the proposed composite sleeper was also conducted. It was found that the composite sleeper satisfied the minimum requirements for flexural performance as reported in the American Railway Engineering and Maintenance-of-way Association (AREMA) and Chicago Transit Authority (CTA) standards.

5.2 Experimental Methods

5.2.1 Material selection

In this research, a glass-fibre pultruded rectangular composite profile having dimensions of 190 mm × 100 mm × 2 m was selected to manufacture the beams. Three beams using
geopolymer concrete filler and three other using normal concrete were cast for comparison purposes. The mix proportions are shown in Table 5.1.

Table 5.1: Mix ingredients for normal and geopolymer concrete

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Normal concrete, (kg/m³)</th>
<th>Geopolymer concrete, (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregate, (SSD)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14 mm</td>
<td>262</td>
<td>233</td>
</tr>
<tr>
<td>10 mm</td>
<td>524</td>
<td>544</td>
</tr>
<tr>
<td>7 mm</td>
<td>352</td>
<td>313</td>
</tr>
<tr>
<td>Fine aggregate, (SSD)</td>
<td>614</td>
<td>468</td>
</tr>
<tr>
<td>Cement</td>
<td>399</td>
<td>-</td>
</tr>
<tr>
<td>Fly ash</td>
<td>-</td>
<td>399</td>
</tr>
<tr>
<td>Sodium hydroxide (NaOH) solution</td>
<td>-</td>
<td>81</td>
</tr>
<tr>
<td>Sodium silicate (Na₂SiO₃) solution</td>
<td>-</td>
<td>185</td>
</tr>
<tr>
<td>Super-plasticiser</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Water</td>
<td>187</td>
<td>-</td>
</tr>
</tbody>
</table>

5.2.2 Preparation of the composite beam

To facilitate the pouring of the concrete, three pultruded hollow profiles were placed on a base-plate to provide better support at the bottom of the beam as shown in Fig. 5.1. The normal concrete beams were cast on the same day using the same mix to minimise variations. A similar process was also used for the geopolymer concrete beams.
5.2.3 Slump testing

The conventional slump test was used to assess the workability of the normal and geopolymer concretes. Both showed good slump values of 90 mm and 150 mm respectively depicted in Fig. 5.2.

5.2.4 Curing of the beams

Since a pultruded FRP profile can encapsulate the moisture within the concrete inside the beam, it was not necessary to keep the normal concrete beams in a highly moisturised room. However, as geopolymer concrete requires heating to accelerate its polymerisation process, a power blanket able to be heated up to 71°C, was used to heat
the geopolymer beams as shown in Fig. 5.3. All the three geopolymer concrete beams were wrapped in this blanket and placed vertically for heating. This heating lasted for 3 days from the day after casting. The geopolymer cylinders were also kept inside the blanket. Since the blanket could not be folded in the longitudinal direction, a heat insulator was used at the tops of the beams. A thermocouple was used to record the interior temperature, which showed 60°C on average during heating.

![Image of heating setup](image)

**Fig. 5.3:** Heating of the geopolymer composite beams using a power blanket

After hardening, all the beams were stored in a temperature controlled room at 23°C with 50% humidity. The beams shown in Fig.5.4 were specifically marked and their corresponding cylinders stored in the same room under sealed conditions until testing.

![Image of stored beams and cylinders](image)

**Fig. 5.4:** Storage of beams and cylinders under sealed conditions
5.2.5 Test set-up and procedure

The six beams were tested along the weak axis in a four-point bending setup to determine the flexural properties such as the bending modulus \( E \) and modulus of rupture \( \text{MOR} \). All the beams have the same span length of 1440 mm that is slightly higher than 1435 mm, which is the standard gauge length of railways in Australia. The schematic diagram for the experimental setup is shown in Fig 5.5.

![Diagram of load, shear and bending moment in static bending](image)

Fig. 5.5: Diagram of load, shear and bending moment in static bending

To measure the mid-span deflection, two linear variable differential transformers’ (LVDT) were placed on each side of the beams. The beams were tested under displacement control at a constant rate of 1 mm/min. All instrumentation and a 32-bit data acquisition system for recording the mid-span load and displacement data at a continuous rate were installed on the day of testing. The test setup arrangements are shown in Fig. 5.6.
5.3 Load-displacement Behaviour

The load-deflection responses at the mid-spans of the tested composite beams are presented in Fig. 5.7. The curves are slightly nonlinear. Ultimate failures of the geopolymer composite beams occurred at loads 91, 99 and 96 kN but, for the normal concrete composite beams, it happened at loads 120, 111 and 115 kN. This was due to the lower strength of 40 MPa of the geopolymer concrete obtained after heat curing using the power blanket. On the other hand, the compressive strength of the portland cement concrete was 57 MPa. As the load increased, failure was initiated in the pultruded FRP composites along the longitudinal direction of the beams and their stiffness slightly decreased from the first stage. Similar observations were noticed by other researchers [116, 117] who studied the behaviour of the concrete-filled Glass-FRP tubes under four-point bending. At the time of failure, composite profile split with the concrete protruding out by up to 1 mm at both ends of the beam shown in Fig. 5.8.
Fig. 5.7: Load-displacement curves for four point bending

Fig. 5.8: Failure of beams: (a) normal concrete; and (b) geopolymer concrete
5.4 Experimental and theoretical flexural rigidities

Since the behaviour of the pultruded profile is generally linear, the slight nonlinearity of the load-displacement curves may be the result from cracks developing in the concrete, which are not visible because the concrete is encased in the pultruded profile. To ascertain this, the theoretical stiffness of the beam is calculated in two ways: non-cracked concrete and a cracked concrete represented in Fig. 5.9.

![Beam section assuming: (a) non-cracked concrete; and (b) cracked concrete](image)

The theoretical flexural rigidity for the beam section can be calculated by the following equation.

**Assuming the concrete is non-cracked and behaves elastically**

From Fig. 5.9 (a),

\[
EI_{(the)} = E_{gc} \left( \frac{(b-2t)(h-2t)^3}{12} \right) + E_p \left( \frac{bh^3}{12} - \frac{(b-2t)(h-2t)^3}{12} \right)
\]  

(5.1)

where,

\[E_{gc} = \text{modulus of elasticity of the geopolymer concrete} = 19000 \text{ MPa};\]  

\[E_p = \text{modulus of elasticity of the pultruded composite} = 28870 \text{ MPa}.\]

\[EI_{(the)} = 3.86 \times 10^{11} \text{ N} \cdot \text{mm}^2\]

**Assuming the concrete is cracked below neutral axis**

From Fig. 5.9 (b),
\[ EI_{(t,he)} = E_g \left( \frac{(b-2t)(y-t)^3}{3} \right) + 2E_p \left[ \frac{t(h-2t)^3}{12} + \left( t \times (h - 2t) \times \left( \frac{h}{2} - y \right)^2 \right) \right] + \]
\[ E_p \left[ \frac{bt^3}{12} + \left( b \times t \times \left( y - \frac{t}{2} \right)^2 \right) \right] + E_p \left[ \frac{bt^3}{12} + \left( b \times t \times \left( h - y - \frac{t}{2} \right)^2 \right) \right] \quad (5.2) \]

\[ EI_{(t,he)} = 2.92 \times 10^{11} \text{ N} - \text{mm}^2 \]

**Flexural rigidity from the experimental results**

In the case of four-point loading, the mid-span deflection is due to pure bending as there is no shear force developed in the middle third. The average slope of the load-displacement curve, \( m = 4.7 \text{ kN/mm} \).

\[ EI_{(exp)} = C_f \times m \quad (5.3) \]

where, the constant, \( C_f = \frac{23L^3}{1296} = 52992000 \text{ mm}^3 \)

or, \( EI_{(exp)} = 2.5 \times 10^{11} \text{ N} - \text{mm}^2 \)

It can be seen that the experimental flexural rigidity is closer to the theoretical when the concrete below neutral axis is assumed cracked rather than when it is assumed non-cracked. Therefore, a cracked section is assumed in all the calculations that follow.

### 5.5 Effective Modulus of Rupture (MOR) of Sleepers

The modulus of rupture (MOR) is defined as the maximum capacity of a member in bending, and can be computed by the bending stress equation [42]. This estimation is essential as the sleepers’ bending performance depends on it.

\[ \text{MOR} = \frac{MC}{I_t} \quad (5.4) \]

where

\[ M = \frac{PL}{6} = \text{the bending moment, presented in Fig. 5.5;} \]

\[ C = \text{the distance of the neutral axis from the outer most fibre of FRP composite;} \]
Here, ‘$h$’ is the depth of the section and the distance of the neutral axis from the top of the section is represented by ‘$y$’.

$I_e$ = the transformed moment of inertia with respect to neutral axis.

The fundamental assumptions relating to flexure stated that the material of the beam should be homogeneous. But in the present case, the beams are non-homogeneous in that they are made of two entirely different materials. Therefore, the flexural analysis should be different from those used in homogeneous beams. The section of beam is considered as a virtual equivalent homogeneous section where the actual area of the FRP composite is replaced with an equivalent concrete area depending on the modulus of elasticity of these two materials presented in Fig. 5.10, 5.11 and 5.12. The equivalent ‘transformed section’ and their moment carrying capacity are discussed in the following section.

**Material Properties**

The modulus of elasticity of normal concrete, $E_{nc} = 30000$ MPa

The modulus of elasticity of geopolymer concrete, $E_{gc} = 19000$ MPa

Longitudinal modulus of elasticity of pultruded FRP composite, $E_p = 28870$ MPa

![Fig. 5.10: Original dimensions of beam section]
**Transformed section for normal concrete beam**

Modular ratio, \( n_{nc} = \frac{E_p}{E_{nc}} = 0.95 \)

Equivalent width of FRP (top and bottom) = \( 190 \times 0.95 = 180 \) mm

Equivalent width of FRP (left and right side) = \( 10 \times 0.95 = 9.5 \) mm

![Diagram of transformed section for normal concrete beam](image)

**Fig. 5.11: Transformed section for normal concrete beam**

Transformed moment of inertia with respect to neutral axis (N.A) before tension cracking of concrete,

\[
I_{tn} = \frac{170 \times 80^3}{12} + \left( 2 \times \frac{9.5 \times 80^3}{12} \right) + 2 \times \left( \frac{180 \times 10^3}{12} + 180 \times 10 \times 45^2 \right)
\]

or, \( I_{tn} = 15384000 \) mm\(^4\)

The calculation for the depth of neutral axis after tension cracking is discussed in section 5.5.3. The transformed moment of inertia with respect to neutral axis (N.A) after tension cracking of concrete,

\[
I'_{tn} = \left( \frac{170 \times 80^3}{12} + 170 \times 80 \times 11.18^2 \right) + 2 \times \left( \frac{9.5 \times 80^3}{12} + 9.5 \times 80 \times 11.18^2 \right) + \left( \frac{180 \times 10^3}{12} + 180 \times 10 \times 33.82^2 \right) + \left( \frac{180 \times 10^3}{12} + 180 \times 10 \times 56.18^2 \right)
\]

or, \( I'_{tn} = 17723858 \) mm\(^4\)
Transformed section for geopolymer concrete beam

Modular ratio, \( n_{gc} = \frac{E_p}{E_{gc}} = 1.50 \)

Equivalent width of FRP (top and bottom) = \( 190 \times 1.5 = 285 \text{ mm} \)

Equivalent width of FRP (left and right side) = \( 10 \times 1.5 = 15 \text{ mm} \)

\[
\begin{align*}
\text{Transformed moment of inertia with respect to neutral axis (N.A) before tension cracking of concrete,} \\
I_{tg} &= \frac{170 \times 80^3}{12} + \left( 2 \times \frac{15 \times 80^3}{12} \right) + 2 \times \left( \frac{285 \times 10^3}{12} + 285 \times 10 \times 45^2 \right) \\
&= 20123333 \text{ mm}^4
\end{align*}
\]

or, \( I_{tg} = 20123333 \text{ mm}^4 \)

Transformed moment of inertia with respect to neutral axis (N.A) after tension cracking of concrete,

\[
\begin{align*}
I'_{tg} &= \left( \frac{170 \times 80^3}{12} + 170 \times 80 \times 9.02^2 \right) + 2 \times \left( \frac{15 \times 80^3}{12} + 15 \times 80 \times 9.02^2 \right) \\
&\quad + \left( \frac{285 \times 10^3}{12} + 285 \times 10 \times 35.98^2 \right) \\
&\quad + \left( \frac{285 \times 10^3}{12} + 285 \times 10 \times 54.02^2 \right) \\
&= 21888854 \text{ mm}^4
\end{align*}
\]

or, \( I'_{tg} = 21888854 \text{ mm}^4 \)
5.5.1 Performance compared with existing composite sleeper

The minimum requirements for the MOR recommended by the American Railway Engineering and Maintenance-of-way Association (AREMA) and Chicago Transit Authority (CTA), and those of other existing composite railway sleepers [12, 41, 42, 112] are presented in Fig. 5.13 together with the average obtained for the present tests. It can be seen that the proposed composite sleeper satisfies the minimum requirements of the AREMA and CTA standards, and also performed better in terms of MOR than the other existing composite sleepers.

![Fig. 5.13: Comparison of MORs with standard and existing composite sleepers](image)

5.5.2 Performance compared with traditional timber sleeper

The MORs of traditional timber sleepers under different conditions measured by Duckworth [108] and Reid [109] are compared with that of the present composite sleeper and shown in Fig. 5.14, which indicates that the flexural strength of the proposed composite sleeper is above the lower value of traditional timber sleepers.

![Fig. 5.14: Comparison of MORs with traditional timber sleepers](image)
5.5.3 Performance compared with concrete sleeper

Australian standard AS 1085.14 considers the rail seat load for the design of concrete sleeper to be:

\[ R = j \times Q \times \frac{D_F}{100} \]  \hspace{1cm} (5.5)

where \( j \) is the design load factor (2.5), \( Q \) is static wheel load (125 kN), and \( D_F \) is the axle load distribution factor (52% for 610 mm spacing, presented in Fig. 3.2).

Therefore, the design rail seat load is obtained as \( R = 163 \text{ kN} \).

If the proposed composite railway sleepers are placed in the real track with standard gauge length (1435 mm) and a sleeper spacing of 610 mm, the bending moment developed at the mid section of sleeper can be obtained from the uniformly distributed ballast pressure under the sleeper according to AS 1085.14. The schematic diagram is shown in Fig. 5.15.
The moment developed at the centre of the sleeper can be calculated by

\[ M_c = \frac{R(2g-l)}{4} \]  

(5.6)

where ‘\( g \)’ and ‘\( M_c \)’ are the gauge length and moment developed at centre of the sleeper due to rail seat load (\( R \)) respectively.

Now, let’s check the sectional moment capacity of the proposed composite sleeper.

**Sectional moment capacity of the proposed composite sleeper**

The sectional analysis is based on the following assumptions;

(a) A cross section that was plane before bending remains plane after bending;
(b) Concrete in tension is ignored and the concrete above the neutral axis is under a uniform compression stress. Although at moderate loads, the concrete stress distribution is linear, starting at zero from the neutral axis and increasing linearly in compression zone. But when the load is still further increased, the stress distribution loses its linearity and becomes totally nonlinear at ultimate load which is equivalent to rectangular stress distribution as depicted in Fig. 5.16.

Strain at different level can be calculated as

\[ \varepsilon_1 = \frac{\varepsilon_c y}{(y - t)} \]
\[ \varepsilon_2 = \frac{\varepsilon_c}{(y - t)} (h - y - t) \]
\[ \varepsilon_3 = \frac{\varepsilon_c}{(y - t)} (h - y) \]

Where, the force can be represented by

\[ F_{p1} = \frac{E_p \varepsilon_c (\varepsilon_1 + \varepsilon) b t}{2} \]
\[ F_{p2} = \frac{E_p \varepsilon_c}{2} [2(y - t)t] \]
\[ F_{p3} = \frac{E_p \varepsilon_2}{2} [2(h - y - t)t] \]
\[ F_{p4} = \frac{E_p (\varepsilon_2 + \varepsilon_3) b t}{2} \]
\[ F_c = \alpha f_{ct} \beta (y - t)(b - 2t) \]

Here, \( y \) = depth of neutral axis from the top which can be calculated from the static equilibrium condition.

\[ \sum F_{tension} = \sum F_{compression} \]
Now, it is obtained that, $\varepsilon_1$ and $\varepsilon_3$ both are less than the ultimate tensile strain of pultruded FRP composite, $\varepsilon_p = 0.011$.

The location of each force from the top of the section is obtained as follows:

$$y_{Fp1} = \frac{(\varepsilon_1 + 2\varepsilon_c)t}{3(\varepsilon_1 + \varepsilon_c)}$$

$$y_{Fp2} = t + \frac{(y - t)}{3}$$

$$y_{Fp3} = y + \frac{2}{3}(h - y - t)$$

$$y_{Fp4} = h - \frac{(\varepsilon_3 + 2\varepsilon_2)t}{3(\varepsilon_3 + \varepsilon_2)}$$

$$y_{Fc} = t + \frac{\beta(y - t)}{2}$$

The internal moment developed at the failure of concrete in compression can be calculated as:

$$M_{int} = -F_{p1}(y_{Fc} - y_{Fp1}) - F_{p2}(y_{Fc} - y_{Fp2}) + F_{p3}(y_{Fp3} - y_{Fc}) + F_{p4}(y_{Fp4} - y_{Fc})$$

Mertol et al. [118] used the following relationship for $\alpha$ and $\beta$ depending on 28-day compressive strength of concrete.

$$\alpha = \begin{cases} 0.85 & \text{for } f'_c \leq 69 \text{ MPa} \\ 0.85 - 0.0029(f'_c - 69) & \text{for } f'_c > 69 \text{ MPa} \end{cases}$$

$$\beta = \begin{cases} 0.85 & \text{for } f'_c \leq 28 \text{ MPa} \\ 0.85 - 0.007252(f'_c - 28) & \text{for } f'_c > 28 \text{ MPa} \end{cases}$$

**Theoretical ultimate capacity for normal concrete beam**

Considering, $b = 190 \text{ mm}$, $h = 100 \text{ mm}$, $t = 10 \text{ mm}$, $\varepsilon_c = 0.003$ and $f'_c = 57 \text{ MPa}$

The coefficient $\alpha$ and $\beta$ should be

$$\alpha = 0.85 \text{ and } \beta = 0.65$$

The depth of neutral axis from the top of the section, $y = 38.82 \text{ mm}$,
Therefore, theoretical moment carrying capacity of the section is,

\[ M_{\text{int}} = 24.03 \, kN - m \]

Ultimate failure load can be obtained by equating the internal and external moment which is;

\[ M_{\text{int}} = \frac{P_{\text{the}} \times L}{6} \]

where \( L \) is the span length of the beam (shown in Fig. 5.5) = 1.440 m.

or, theoretical ultimate load, \( P_{\text{the}} = 100.14 \, kN \)

**Theoretical ultimate capacity for geopolymer concrete beam**

Considering, \( b = 190 \, mm, h = 100 \, mm, t = 10 \, mm, \epsilon_c = 0.003 \) and \( f^\prime_c = 40 \, MPa \)

The coefficient \( \alpha \) and \( \beta \) should be

\( \alpha = 0.85 \) and \( \beta = 0.76 \)

The depth of neutral axis from the top of the section,

\( y = 40.98 \, mm, \)

Therefore, theoretical moment carrying capacity of the section is,

\[ M_{\text{int}} = 21.25 \, kN - m, \] which is above the bending moment developed in the real railway track, \( (M_c = 15.08 \, kN - m). \)

Ultimate failure load can be obtained by equating the internal and external moment which is;

\[ M_{\text{int}} = \frac{P_{\text{the}} \times L}{6}, \] where \( L \) is the span length of the beam = 1.440 m.

or, theoretical ultimate load, \( P_{\text{the}} = 88.53 \, kN \)

Therefore, theoretical moment capacity of the proposed geopolymer composite sleeper is above the moment developed at the centre of the sleeper in a real railway track.
5.6 Effective Modulus of Elasticity (E) of Sleepers

In the design of a composite structure, stiffness is considered one of its important parameters along with strength. Stiffness is measured in terms of an equivalent modulus of elasticity of the structure in the elastic ranges of stress and strain. The modulus of elasticity in bending is required to quantify the behaviour of sleeper being deformed under mechanical stress. It is calculated using the static mechanics of the four-point bending test as;

\[
E = \frac{23 \times \frac{P}{2} \times L^3}{648 \times I_t \times \Delta_m}
\]

or, \( E = \frac{23pl^3}{1296I_t\Delta_m} \) \hspace{1cm} (5.7)

where

\( E \) = the flexural modulus of elasticity of the beam (MPa);
\( P \) = the total load on the beam acting in both load points (N);
\( L \) = the span length between the two supports (mm);
\( I_t \) = the transformed moment of inertia with respect to neutral axis; and
\( \Delta_m \) = the deflection at the mid-span (mm).

The equivalent modulus of elasticity of the proposed sleeper is calculated considering its mid-span deflection and corresponding load. Before concrete cracking, the transformed moment of inertia was calculated with respect to the centroidal axis while it was computed about the shifted neutral axis after tension cracking. All the beams get stabilized after taking a few kilo-newtons of load. Variations in its flexural rigidity (EI) with displacement is plotted after getting stabilized the beam which given in Fig. 5.17.
Fig. 5.17: Variations in flexural rigidity (EI) with mid-span displacement

Fig. 5.18 compares the equivalent modulus of elasticity of the proposed alternative composite sleeper with those of the other existing composite sleepers [12, 41, 42, 112] which shows that it satisfied the minimum requirements of the Chicago Transit Authority standard and was well above those of the other existing composite sleepers.

Fig. 5.18: Comparison of flexural modulus of elasticity with standard and others
5.7 Conclusions

The technical procedure and testing methods for determining the static performances of composite railway sleepers were studied experimentally, and it was shown that the proposed composite beam satisfied the minimum flexural requirements for composite railway sleepers stated in the AREMA and CTA standards and also showed satisfactory performance when compared with existing railway sleepers. It was also found that the ultimate failure occurred in the pultruded profile along its longitudinal direction.
Chapter 6

Conclusions and Future Recommendations

6.1 Summary
The unexpected deterioration of existing sleepers under different conditions, as well as concern regarding the huge amount of CO₂ emitted into the environment by the cement and steel industries, are the main drivers for this research which looks at alternative materials for railway sleepers. Pultruded FRP composites and fly ash-based geopolymer concrete are materials used in this research as alternatives to existing materials. The static performance of a composite beam made from these two materials has suggested that introducing this novel, environmentally friendly, composite railway sleeper to the railway industry is possible. Some recommendations facilitating further investigations into the proposed composite railway sleeper are provided.

6.2 Major Conclusions of the Study
6.2.1 Development of a suitable geopolymer concrete for composite sleepers
In this research, the limitations of the currently available mix design procedures suggested by Lloyd and Rangan in 2010 for fly ash-based geopolymer concrete have been focused on, and a rigorous design method is proposed, from which the following conclusions have been drawn.

- This study described a design procedure for fly ash-based geopolymer concrete in general, illustrated it using a flowchart and then provided an example of how it works. Variable concrete densities, the effects of the ingredients’ specific gravities, contributions of air volume, flexibility to improve the workability of fresh concrete and the opportunity to use aggregates in their ‘as is’ condition
were considered important for overcoming the main limitations of current design methods.

- A range of mixes were made to test the proposed method using various water-to-geopolymer solids ratios and different amounts of fly ash content. Experimental results showed that the compressive strength of the fly ash-based geopolymer concrete decreased linearly with increases in the water-to-geopolymer solids ratio. This observation was in agreement with the basic principles of ordinary portland cement concrete, the strength of which decreases with increases in the water-cement ratio.

6.2.2 Experimental investigation into static behaviour of composite sleepers

This study investigated the load-deflection behaviour and failure mechanism of a composite railway sleeper under four-point bending testing. Based on the results of this investigation, the following conclusions are drawn.

- Failure in the pultruded FRP profile propagated along its longitudinal direction, and the end of the composite profile split at the time of ultimate failure with the concrete inside it protruding out by up to 1 mm at both ends of the beam due to slip.

- The slight nonlinearity of the load-displacement curves were obtained from the result of tension cracks in the concrete below neutral axis. These cracks were not visible because the concrete was encased in the pultruded profile. To ascertain this, the theoretical stiffness of the beam was calculated considering both the concrete cracked and non-cracked below neutral axis where the cracked concrete section showed good agreement with the experimental results.
• The modulus of rupture of the proposed composite sleeper satisfied the minimum requirements of the AREMA and CTA standards and also performed better than those of the existing composite sleepers.

• The proposed composite sleeper showed good flexural performances in terms of its modulus of rupture when compared with that of a traditional timber sleeper.

• The sectional analysis of the composite beam showed that its moment carrying capacity was well above that which develops in a real railway track.

• The flexural stiffness of the composite beam in terms of its flexural modulus of elasticity satisfied the minimum requirements of the CTA standard and was comparatively better than those of existing composite railway sleepers.

6.2.3 Theoretical prediction for ultimate capacity of composite sleepers

A simple equation was established for the ultimate moment to predict the approximate failure loads of the beams. The conclusions drawn from the theoretical prediction and experimental investigation of the composite beams are summarised below.

• For the normal and geopolymer concrete beams, the ultimate failure loads were predicted to be 100 kN and 89 kN, respectively. The experimental investigation showed that, on average, the normal concrete beam failed at 115 kN whereas the average ultimate failure load was 95 kN for the geopolymer concrete beam, which is only 15% and 7% respectively, higher than the theoretical predictions.

• The concrete inside the profile provided internal support to prevent local buckling, and contributed to the inertia of the section and internal forces which could increase the flexural strength of the beam.
6.3 Possible Areas for Future Research

Although the static behaviour of a geopolymer composite beam was studied, future challenges for research into, and the development of, composite sleepers still remain. Prior to the widespread application of geopolymer composite beams for railway sleepers, the following aspects need to be studied in more detail.

6.3.1 Investigations into impact behaviour of composite beam

Impact load testing is required to measure the capacity of the proposed sleeper to absorb a shock which may occur while a train is running over it due to abnormalities of either the train’s wheels or the rail, e.g., a flat wheel, dipped rails, etc. Such an imposed load is of a very high magnitude but short duration, and the sleepers’ surfaces should remain uncracked when subjected to two wheel drops of 500 kg each from 75 cm above at the same location [112].

6.3.2 Investigations into dynamic fatigue behaviour

The structural integrity of the proposed product should be assessed by dynamic fatigue testing to ensure the absence of any void and other inherent manufacturing defects, such as dry patches, resin-rich areas, de-lamination, etc., particularly for FRP composite sleepers. A sleeper should not develop any cracks on its surface when subjected to 2 million cycles of dynamic loading, varying from 4 to 20 tonnes vertically and 1.6 to 8 tonnes (i.e., 40% of the vertical load) horizontally at a frequency of 5 Hz.

6.3.3 Fastener electrical impedance test

Unless a sleeper is electrically resistant to minimise problems for signalling, special care is required in track-circuited areas. According to the AREMA specification, the minimum required impedance is 10,000 ohms when a wetted sleeper is subjected to 10 volts AC 60 Hz between two rails for a period of 15 minutes.
6.3.4 Thermal expansion and contraction test

The expansion and contraction of a composite sleeper due to changes in temperature should be measured as they have an effect on a railroad’s gauge length. Union Pacific has provided a simple procedure for measuring this parameter in which they used a thermal cycle $(23^\circ C) \rightarrow (71.1^\circ C) \rightarrow (-17.7^\circ C) \rightarrow (23^\circ C)$ with a period of 24 hours in each stage [119].

6.3.5 Spike insertion-withdrawal and screw withdrawal force test

To measure the rail gauge and rollover restraint capacity of a sleeper, this test is normally carried out and can be done by measuring the force required for the insertion and withdrawal of a spike as well as the withdrawal of a screw.

6.3.6 Finite element modelling of static and dynamic testing

To predict the experimental behaviour of a composite beam, a finite element analysis could be performed as well as an experimental investigation. Different finite element packages are available in the market, with ANSYS and ABAQUS the most commonly used for research purposes.
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