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Influence of Waste Tire Rubber Particles Size on the Microstructural, Mechanical, and Acoustic Insulation Properties of 3D-Printable Cement Mortars

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Abstract

3D printing technologies of construction materials are gaining ground in the building industry. As well documented in the literature, these advanced manufacturing methodologies aim to reduce work-related injuries and materials waste, enhancing architectural flexibility which would enable more sophisticated designs for engineering and aesthetic purposes. In this framework, the development of functional and eco-sustainable printable materials represents an extremely attractive challenge for research, promoting digital fabrication to reach its maximum cost-effective and technological potentials. The use of recycled tire rubber particles in 3D printable Portland-based compounds is an exclusive contribution in this field. This line of research aims to integrate the well-known engineering performances of rubber-cement materials with the advanced peculiarities of additive manufacturing methodologies. As an innovative contribution, the authors propose here a detailed study on the possible relationship between rubber particle size and technological properties of the 3D printable mix. Specifically, two groups of continuous size grading polymer aggregates (0-1 mm rubber powder and 1-3 mm rubber granules as fine and coarse fractions, respectively) were analyzed in terms of impact on rheology, print quality, microstructure, mechanical properties, and acoustic insulation performance. Concerning the print quality, rubber aggregates altered the fluidity of the fresh mix, improving the adhesion between the printed layers and therefore enhancing the mechanical isotropy in the post-hardening sample. A remarkable influence of the rubber gradation on the compounds’ behaviour was found in hardened properties. By comparing the rubberized compounds, the fine polymer fraction shows greater interfacial cohesion with the cement paste. However, more significant mechanical strength loss was found due to a greater reduction in density and increased porosity degree. On the other hand, mortars doped with larger rubber particles tend to have a higher unit weight, finest pore distribution, minor mechanical strength drop, and higher ductility but worse interface binding with the matrix. Regarding the acoustic insulation properties, a proper balance between rubber powder and granules in the mixes allows to obtain comparable/superior performance compared to plain mortar but the effect of the aggregate size is strongly dependent on the sound frequency range investigated. Future findings revolve around applicability studies of these formulations in civil and architectural fields, benefiting from the design flexibility of 3D printing.

Keywords: 3D Printing; Tire Recycling; Rubber Particle Size; Print Quality; Microstructural Investigation; Mechanical Properties; Sound Insertion Loss.

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1. Introduction

Digital manufacturing, such as 3D printing, is one of the most advanced fabrication methodologies in the field of the construction industry, with prospects of revolutionizing the civil and architectural sectors. In the past few decades, a rise in interest towards 3D printing techniques can be mainly attributed to several technological-environmental benefits, listed below [1-4]:

- Targeted and optimized procurement of building materials;
- Wastage reduction;
- Reduction of dangerous manual work and injuries on the construction site;
- Workforce reconversion: traditional labor will be replaced by highly skilled operators programming the printer machines from safe office spaces;
- More architecture freedom: this technology permits design for structural optimization because of its ability to create complex structures without additional labor and cost.

Currently, the extrusion-based printing process is the most explored additive manufacturing option in academia and industry [1, 5]. Similarly to fused deposition modeling (FDM), a visco-plastic cementitious material is extruded out of a deposition nozzle that prints the cement layers. The nozzle is mounted to a robotic motion system and its shape and size are closely related to the characteristics of the desired structure and the size of the aggregates. A pumping system allows the flow of printable material from the mixing unit to the extruder, where it is subsequently released. Starting from a 3D CAD model, the final object is built layer-by-layer using the pre-defined coordinates and the given printing parameters (deposition rate, infill, and layer thickness). Mixture’s rheology is a crucial aspect in obtaining a high-quality printing process and a hardened material free of structural defects. Extrudability (the ability of the material to be deposited regularly and without interruptions in the extrusion system), buildability (the ability of the printed layers to hold the subsequent layers on top of them without collapsing phenomena), and inter-layer adhesion (internal compaction of the material due to the correct cohesion between the printed layers) are considered key indicators about the print quality. In this regard, the fluidity and stiffness of the compounds should be properly balanced to meet the requirements described above [1, 5-6].

To enhance the sustainability of 3D printing processes in the construction sector, an attractive challenge for researchers concerns the development of “green” printable cementitious materials based on the use of industrial waste as raw materials to replace traditional virgin aggregates [4, 7]. The present research aims to investigate the printability requirements, the physical-mechanical properties, and the acoustic insulation behaviour of rubber-cement mortars (RCMs) obtained by total replacement of sand with polymer aggregates deriving from end-of-life tires (ELTs). The replacement of mineral materials by recycled rubber in the printable material creates a new avenue for ELTs and reduces the demand for natural resources. As reported by Roychand et al. [8], 1 billion/year waste tires are accumulated annually, of which >50% are illegally disposed or subjected to highly polluting treatments. This trend has encouraged governments around the world to promote the recycling of ELTs through grinding treatments aimed at obtaining new raw materials. According to European Tyre Recycling Association (ETRA) draft model [9], since 2016, over 1.25 tons/year of tire-derived raw materials were used in various industry proposals, reducing reliance on landfills at a low of 5%. Possible uses of ground rubber include molded products [10], rubberized asphalts with improved acoustic and durability performances [11], and rubber-cement compounds for civil-architectural applications [8]. The inclusion of elastomeric fillers in cementitious matrices may help produce low self-weight structures with cost sustainability, improve the mechanical ductility, enhance the vibro-acoustic damping and the heat insulation behaviour, and increase the chemical-physical durability in terms of porosity reduction, water absorption, and carbonation inertia. However, mechanical strength reduction, due to the poor rubber-cement interface adhesion, represents the weak point of this technology [12-14]. Although many studies on pre-cast concrete mixes modified with recycled tire rubber were proposed, there are no attempts and research activities available on the application of these polymer aggregates in the production of cementitious compounds for 3D printing methods. The objective is to combine the technological functionalities conferred by rubber with the performance of additive manufacturing in terms of design flexibility, structural optimization, and low pollution. Several efforts were already conducted by the authors on this topic, involving preliminary printability and print quality studies to explore the potential of incorporating recycled tire rubber particles into printable cementitious mixes [5, 7, 13], experimental characterization on durability analysis [13], the effect of rubber-based modification on thermal insulation properties [7, 15], first mechanical investigations and modelling [5, 7, 15, 16]. From the perspective of rational production of secondary raw materials by the tire recycling companies, economic impact and material performance, the novel purpose of the present study is to deepen in detail the influence of rubber particle size on the fresh and hardened properties of printable mortars. The manufacturing cost and energy consumption of the tire grinding process vary greatly depending on particle size gradation. These aspects must be carefully considered to encourage the sustainability of the entire
production cycle, and therefore an evaluation of the interaction between rubber particle size and material performance can provide valuable information for both manufacturers and researchers.

Specifically, in this work, the effect of rubber particle size on the rheological, microstructural, mechanical, and acoustic properties of RCMs was investigated. 0-1 mm rubber powder (RP) and 1-3 mm rubber granules (RG) were used as fine and coarse aggregates in the mixes, respectively. First, a detailed characterization of the polymer aggregates is reported to investigate the influence of the gradation on the fresh properties of the mixes and the physico-mechanical behaviour of RCMs. Then, the suitability of proposed compounds for 3D printing fabrication was validated by printability tests, consisting of the manufacture of 6-layers slabs and subsequent assessment of specific print quality requirements. Finally, to evaluate how the rubber-based modification affects the mortars’ features, a series of experimental analysis, including, scanning electron microscopy (SEM), pore size analysis by optical microscopy (OM), cube compressive strength, flexural test, and sound insertion loss (SIL) measurements were conducted on specimens extracted from the printed slabs. Although some similarities with previous publications in the literature, this manuscript presents a more exhaustive and rigorous description of the mechanisms underlying the relationship between rubber characteristics (gradation, morphology, replacement level) and resulting material behavior.

2. Materials and Methods

2.1. Tire Rubber Aggregates

The waste rubber particles, obtained by ambient mechanical grinding of ELTs, were sourced from European Tyre Recycling Association (ETRA, Belgium) plants. Two different polymer aggregates were investigated in this work: RP (Figure 1a) which was a fine fraction with grading close to the limestone sand used in this research, and RG (Figure 1b) as coarse fraction.

![Figure 1. Rubber particles used in this work: (a) 0-1 mm RP; (b) 1-3 mm RG](image)

Experimental sieving analysis (Figure 2 and Table 1), performed in accordance with DIN 66165 standard method [17], revealed 0.425-1 mm range size for RP and 1.7-3 mm range size for RG.

![Figure 2. Grading curves of tire rubber aggregates](image)
Table 1. Sieving analysis results of tire rubber aggregates

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>% passing</th>
<th>Sieve size (mm)</th>
<th>% passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>99.70</td>
<td>3</td>
<td>99.13</td>
</tr>
<tr>
<td>0.710</td>
<td>66.94</td>
<td>2</td>
<td>51.28</td>
</tr>
<tr>
<td>0.425</td>
<td>16.55</td>
<td>1.7</td>
<td>22.47</td>
</tr>
<tr>
<td>0.125</td>
<td>0.28</td>
<td>1</td>
<td>1.35</td>
</tr>
</tbody>
</table>

The average specific gravity of rubber particles, measured by Micromeritics AccuPyc 1330 He-pycnometer, is 1.2 g/cm³. The morphology and surface texture of rubber particles were inspected by scanning electron microscopy (SEM) analysis. Prior to the examination, rubber samples were fixed on a carbon adhesive tape and then made conductive by graphitization treatment using a Leica EM SCD005 vacuum sputter coater. Secondary-electron (SE) imaging (Figure 3) were collected using a Tescan Mira3 FEG-SEM.

The morphology and surface texture of rubber particles were inspected by scanning electron microscopy (SEM) analysis. Prior to the examination, rubber samples were fixed on a carbon adhesive tape and then made conductive by graphitization treatment using a Leica EM SCD005 vacuum sputter coater. Secondary-electron (SE) imaging (Figure 3) were collected using a Tescan Mira3 FEG-SEM.

From SEM analysis, it is possible to notice the different surface morphology of the aggregates: RP shows rough and irregular texture, while in RG a smoother and more regular surface is detectable. This is attributable to the shredding treatment. To obtain a greater fineness degree, more grinding cycles are required, resulting in more irregular shaping of the rubber particles [16].

2.2. RCM Mixtures Proportioning

The rubberized compounds were obtained from plain Portland-based mortar (designated as “CTR”) by total volume replacement of the sand with the polymer aggregates. CTR mix is composed of Type I Portland cement (800 kg/m³), limestone sand of 0.4 mm maximum size (1100 kg/m³), and water (300 kg/m³).

Three RCMs (designated as “RP100”, “RP50-RG50”, and “RP25-RG75”) were developed by incorporating the rubber fractions in different proportions: RP100 refers to mix with 100% by volume of RP, RP50-RG50 consists of equal volume content of RP and RG, and RP25-RG75 provides RP and RG in 1:3 proportion ratio. The water amounts were adjusted by trial printability tests aimed to evaluate the correct extrudability and buildability properties of the fresh mixes. The water contents in the rubber-based mortars (260, 250, and 230 kg/m³ in RP100, RP50-RG50, and RP25-RG75, respectively) are lower than CTR mix. According to Lyse’s rule and the typical workability behaviour of cementitious compounds, the larger size and more regular shape of rubber aggregates than sand imply a lower water dosage to reach the same fresh material consistency [13, 18]. This also explains the decreasing trend in the water content when RG content is added in the mixes.

Finally, all the investigated mixes had a fixed content of chemical admixtures (152 kg/m³) to ensure suitable rheology for the printing process. The correct balance of additives is summarized in Table 2.
Table 2. Admixtures content in printable mortars

<table>
<thead>
<tr>
<th>Admixture</th>
<th>Amount</th>
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</thead>
<tbody>
<tr>
<td>Silica fume-based thixotropic additive</td>
<td>79 %</td>
</tr>
<tr>
<td>Polycarboxylate ether-based superplasticizer</td>
<td>2.6 %</td>
</tr>
<tr>
<td>Aliphatic-based water reducing agent</td>
<td>5.2 %</td>
</tr>
<tr>
<td>Calcium oxide-based expansive agents</td>
<td>13.2 %</td>
</tr>
</tbody>
</table>

2.3. Specimens Manufacturing and Testing Methods

A custom-made printing system was used to produce the mortar samples for the materials characterization. The apparatus involves a Comau Robotics 3-axis robotic arm (Figure 4a) equipped with a PVC circular nozzle (Ø = 10 mm). The nozzle is connected to a pressure vessel (4 bar) containing the fresh mix, which is extruded with a constant deposition speed of 33 mm/s. Moderate external vibration was applied to the vessel to ensure adequate compaction of the cement compounds. Starting from a 3D CAD model, 6-layers slabs (230×160×55 mm) were printed for each mix. Ultimaker Cura slicing software was used to generate the GCode file of the object and select the main process parameters. After the printing, the slabs (Figure 4b) were cured for more than 28 days at ambient temperature and, therefore were cut by a diamond circular saw to obtain four types of specimens for the laboratory tests.

![Image of 3D printing system](image1)

![Image of rubber-cement slab](image2)

Figure 4. (a) 3D printing system; and (b) rubber-cement slab after 28 days of curing

A flowchart of this research methodology is presented in Figure 5.

![Flowchart of research methodology](image3)

Figure 5. Flowchart of the research methodology conducted in this work

2.3.1. Bulk Density Testing Method

Bulk density (ρ) measurements were conducted in accordance with BS 1881-114 standard method [19] on 48x42x22 mm blocks. After 48 hours of oven-dry treatment (110 °C), the weight and the geometric volume of the samples were determined, then ρ was calculated as mass to volume ratio. For each mix, ρ-result is an average value of four tested specimens.

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2.3.2. Mechanical Characterization

To evaluate the compressive strength (σc) of printed mortars, 40 mm-side cubes were tested in two different load directions, namely Z-direction (load direction perpendicular to printed filaments plane) and X-direction (load direction longitudinal to printed filaments plane). Figure 6a illustrates the orientations of the compressive load with respect to the layered structure of the printed element. Mechanical tests were performed in accordance with ASTM C109/C109M-20a [20], using a Zwick-Roell Z150 machine (150 kN capacity) at a loading rate 1 mm/min. For each loading direction three samples were analysed. A detail of the test is shown in Figure 6b.

Four-point flexural test was conducted following the ASTM C348-20 standard method [21] to measure the flexural strength (σf), the Young’s modulus (E), and the elongation at break (ε) of investigated mortars. 40×40×220 mm beams were tested by a Zwick-Roell Z010 machine (10 kN capacity), using a support span of 180 mm at a loading rate of 1 mm/min. Contrary to compressive test, the specimens were tested by applying the bending load orthogonally to the print plane (Z-direction). Three beams per mix were mechanically investigated under flexural load.

![Figure 6](image_url)

Figure 6. Compressive test on rubber-cement specimens: (a) Spatial orientation of the compressive loads; (b) Experimental configuration

2.3.3. Acoustic Insulation Analysis: Sound Insertion Loss Measurements

The influence of rubber aggregates on acoustic insulation properties of printable mortars was investigated by sound insertion loss (SIL) analysis. The experimental test was performed by using a two-microphone impedance tube (Figure 7) in accordance to ISO 7235 standard method [22]. The tube had an internal diameter of 16 cm, a length of 100 cm, and mounted two ¼” condenser microphones (Behringer ECM800) at a distance of 40 cm for sound insulation measurements. A Behringer MPA30BT loudspeaker, at one end of the tube, generated a Log sweep acoustic signal from 50 Hz to 4000 Hz. The loudspeaker was housed inside an acoustic box filled with sound-absorbing polyurethane foam to ensure the correct sound flow in the duct. A Focusrite Scarlett 2i4 audio interface was used for the processing of audio signals during the experiment. The other end of the tube is sealed with an absorbent termination to minimize unwanted acoustic reflections in the tube. 80 mm x 80 mm x 40 mm blocks were placed in the middle part of the duct between the microphones. Experimental evaluation of SIL-index consists in measuring the sound pressure level gradient between the two microphones with and without the specimen. SIL as a function of the acoustic frequency (f) is calculated as follows (Equation 1):

\[
SIL(f) = \Delta L_0(f) - \Delta L_s(f)
\]

Where:

- \(SIL(f)\) is the sound insertion loss as a function of the acoustic frequency [dB];
- \(\Delta L_0(f)\) is the sound pressure level gradient between the microphones without the sample [dB];
- \(\Delta L_s(f)\) is the sound pressure level gradient between the microphones with the sample [dB].

For all four investigated mortar, SIL-values in the range 50-4000 Hz was plotted. The data were acquired and processed by Room EQ Wizard software.
2.3.4. Microstructural Analysis

Scanning electron microscopy (SEM) investigation was conducted to analyse the effect of particle size on the rubber-cement interface properties of the RCMs. Small fragments of investigated materials (about 5 g) were examined with a Tescan Mira3 FEG-SEM. The specimens were coated with graphite prior to the analysis using the same procedure performed on the granular tire rubber samples (see Section 2.1).

A specific methodology and software were used to investigate the pore structure via digital image analysis. The same test specimens used in ρ measurements were analyzed by a Leica MS5 stereomicroscope. For each formulation, four cross-section images (16x magnification) were acquired by using Lucia Imaging software. The micrographs were analyzed by ImageJ software to obtain the surface characteristics and macro-porosity (100μm-1 cm), of the specimens. Considering a representative number of voids, pore size distribution was evaluated by measuring the effective diameter of each void, assuming it to be perfect circle. The corresponding cumulative frequency (based on the number of air pores detected) was computed, according to the following size ranges: ≤100 μm; 100-200 μm; 200-300 μm; ≥300 μm.

3. Results and Discussions

3.1. Print Quality Investigation

Mortars’ print quality was evaluated considering three printability indicators: a) regular and uninterrupted extrusion of the fresh mix (extrudability); b) layer-by-layer stacking capability without collapse (buildability); c) printed filaments cohesion (inter-layer adhesion). The 3D-printable compounds developed in this study satisfied the abovementioned requirements. However, as can be seen in Figure 8a, all the RCMs exhibited greater structural compaction than CTR mix. As confirmed by previous studies [13, 23], an influence of adding polymer aggregates into cementitious compounds is the improvement in fluidity. The hydrophobic groups in the macromolecular structure of rubber could attract air in the cement paste and incorporate air bubbles during the mixing, by reducing the overall surface tension of the fresh mix. This phenomenon made the paste flow more easily, enhancing the inter-filament bond in the RCM samples. As a consequence of this evidence, the air-entraining performance of rubber is inevitably crucial about specific physical-rheological properties, including viscosity, pore structure, and unit weight [24].

Furthermore, the rubber particles are less water-absorbent than the mineral aggregates, leading to a higher rate of free water in the fresh mixes. During the printing process, surface moisture acts as a cohesive layer for the printed filaments, improving the intermixing of the layers and interfacial adhesion. In the context of extrusion-based digital fabrication, layer-to-layer adhesion is a major factor affecting the integrity of a 3D-printed structure.

In Figure 8b, it is possible to observe a more evident stratification in CTR sample compared to the rubberized one (in this example RP100 sample), where a more homogenous structure occurs. The weak interfacial bond, generally defined as “cold joint”, induces pronounced mechanical anisotropy, strength loss, and negatively affects the durability performance of the printed element [25]. In this respect, modification with rubber has proved to be an efficient strategy for improving the structural quality of the material.
3.2. Microstructural Analysis

The interfacial binding efficiency between the rubber aggregates and surrounding cement matrix is shown in the 1kX magnification SEM SE-micrographs (Figure 9) reported below.

As mentioned earlier, the mechanical grinding degree affects the surface morphology of tire rubber particles. As shown in Figure 7b, the coarser polymer fraction has a smaller specific surface area than the finest one, resulting in a weaker cohesion with the cement and porous Interfacial Transition Zones (ITZs). Conversely, the presence of structural “jaggies” in RP, due to their irregular texture, is beneficial about the adhesion with the cement matrix. The relationship between rubber particle morphology and adhesion properties with the cement paste agree the research of Ghizdăveț et al. [26].

Air bubbles content in cementitious materials results from several factors: accidental incorporation of air in the fresh mix, incomplete compaction of the cement paste, or excessive water dosage. When the material hardening occurs, the air bubbles turn into mechanical weakness elements, causing structural defects and degradation points [27]. Macro-pores and capillary porosity (<10 μm) govern numerous properties of concrete and mortars, such as mechanical strength, elasticity, and permeability [28].

Optical microscopy analysis revealed interesting effects of rubber inclusion on the microstructural characteristics of the cementitious compounds. The major findings are listed below:

- **Air bubbles porosity reduction.** From the optical micrographs (Figure 10), it is possible to qualitatively observe an overall surface porosity reduction in the rubberized compounds compared to CTR sample. The alteration in rheological behaviour when the rubber is incorporated into the cement mixture is the main reason for this trend. First, as well pointed by Sun et al. [29], rubber modification reduces the viscosity and the flow resistance of the
cement paste. Low plastic viscosity promotes the diffusion and the escape of air bubbles from the paste, preventing their retention in the hardening material [30]. Besides, rubber aggregates reduced water content in the cementitious mixes, maintaining proper fresh properties for printing (see Section 2.2). This effect is more pronounced in RG-based mortars, where the concentration of air voids tends to gradually decrease. The influence of water dosage on the material porosity is supported by several literature pieces of research [24, 26]: with the decrease in the water-to-cement (w/c) ratio, the rate of free water that evaporates is lower, implying a reduction in air voids rate.

![Figure 10](image1.png)

**Figure 10. Microstructural analysis by optical microscopy: CTR (a), RP100 (b), and RP25-RG75 (c) samples**

- **Pore size reduction.** The influence of rubber on w/c ratio also plays a key role on the pores dimension. This evidence is well demonstrated by the graph in Figure 11. Which highlights the pore size distribution evaluated by imaging analysis.

![Figure 11](image2.png)

**Figure 11. Cumulative pore size distribution for CTR and rubberized mixes**

The pores size of CTR sample (w/c ratio = 0.375) is homogeneously distributed in the range from 100 μm up to more 300 μm, with negligible concentration of small voids (< 100 μm). On the other hand, opposite trend can be observed in rubberized mixes. The proportion of pores in RP100 (w/c ratio = 0.325) and RP50-RG50 (w/c ratio = 0.312) is mainly located in 100 – 200 μm range, indicating a less evident contribution of coarse macro-porosity (> 200 μm). The lower w/c ratio formulation, i.e. RP25-RG75 mix (w/c ratio = 0.287), highlighted the predominant contribution of finest porosity. These results are consistent with the literature findings drawn from the relationship between air pores structure and water content in cementitious media [31]. Voids dimension is mainly controlled by water amount: higher w/c ratio results in large pores in the cement matrix. Several studies demonstrated the functionality of finest air bubble porosity on cement materials’ engineering performances. As ascertained by Das and Kondraiven (2019), smaller size pores enhance the durability of concrete in terms of permeability and hydraulic diffusivity inertia. Neithalath et al. [3] revealed strong influence of air void dimensions on the acoustic properties of enhanced porosity concrete. The reduction in pore size increases the maximum acoustic absorption, as for a propagation medium with small sized pores the frictional losses are high and a considerable portion of the acoustic waves that enter the pore system is dissipated. Remesar et al. [34] studied the effect of pore size gradation on the thermal insulation properties of lightweight concrete. At the same porosity degree, smaller pores own many more reflecting/refracting surfaces of heat flux, preventing the radiative transfer into the material and improving its heat insulation.
The rheology and microstructural properties induced by the rubber inclusions are hypothetically explanatory of the higher inter-layer adhesion revealed in the rubberized samples compared to CTR mix. The low water evaporation rate, due to the reduced w/c ratio and the preferential presence of fine porosity, can be considered a key factor regarding the cohesion between the extruded filaments. Indeed, the maintenance of a certain degree of surface moisture in the printed filaments during the additive process allows a better interface bonding, avoiding the premature hardening of the deposited materials and stiffness mismatches that can create interfacial voids (as clearly observed in the CTR sample). The relationship between inter-layer adhesion of 3D printable cementitious materials and surface humidity degree is well documented in Weng et al. research [35].

3.3. Physical-mechanical Characterization

Table 3 reports the results of the experimental programme based on bulk density and mechanical tests.

<table>
<thead>
<tr>
<th>Sample</th>
<th>ρ (kg/m³)</th>
<th>σ_x (MPa)</th>
<th>σ_y (MPa)</th>
<th>σ_z (MPa)</th>
<th>E (GPa)</th>
<th>ε (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTR</td>
<td>1927</td>
<td>63.1</td>
<td>44.1</td>
<td>4.21</td>
<td>5.56</td>
<td>0.05</td>
</tr>
<tr>
<td>RP100</td>
<td>1340</td>
<td>14.5</td>
<td>11.6</td>
<td>1.37</td>
<td>1.94</td>
<td>0.37</td>
</tr>
<tr>
<td>RP50-RG50</td>
<td>1624</td>
<td>18.9</td>
<td>16.8</td>
<td>2.11</td>
<td>1.89</td>
<td>0.40</td>
</tr>
<tr>
<td>RP25-RG75</td>
<td>1468</td>
<td>15.7</td>
<td>11.1</td>
<td>1.58</td>
<td>1.82</td>
<td>0.55</td>
</tr>
</tbody>
</table>

The sand-rubber replacement reduces ρ-values. Unit weight drop can be mainly attributed to the lower specific gravity of the rubber particles than mineral ones, the contribution of ITZ voids, and the variation in the overall porosity depending on w/c ratio [5, 16, 36]. The most relevant density reduction is observable in RP100 mix, where the total incorporation of fine polymeric fraction implies a higher weight content of rubber in the material.

The bulk density trend is in line with the lower σ_x and σ_y in RCMs, which also depend on the weak rubber-cement interface adhesion and the soft properties of polymer fillers, resulting in premature cracking in the surrounding cement matrix. The results of this work are consistent with previous experimental studies [5, 16, 37, 38]. However, two noteworthy findings can be deduced from the compressive strength results.

- Although the sand-rubber replacement level was similar in all the rubberized formulations, by varying the proportion of fine and coarse polymer fraction it is possible to modulate the mechanical strength properties. By reducing the w/c ratio increases the strength of cement matrix around the rubber particles. This explains the lower mean compressive and flexural strength loss in RP50-RG50 (-50% for σ_x, -70% for σ_y, and -62% for σ_z) and RP25-RG75 (-63% for σ_x, -75% for σ_y, and -75% for σ_z) compared to RP100 (-68% for σ_x, -77% for σ_y, and -74% for σ_z). Furthermore, the contribution of the particle size was considered. In general, the strain capacity is significantly enhanced when rubber particles are added and attributed to the ability of polymer inclusion to reduce stress concentration and delay the coalescence and propagation of micro-cracks [39]. The coarser rubber fraction appears to act more efficiently in terms of crack-arresting properties, resulting in better mechanical behaviour. By comparing the “hybrid” rubber-cement formulations, the divergences in mechanical strength are mainly attributable to the rubber-cement interface properties. As previously discussed, RG involve weaker adhesions with the cement paste (ITZ defects), deleteriously affecting the structural integrity of the sample. This explains the lower strength levels in RP25-RG75 mix compared to RP50-RG50.

- By comparing X and Z-directions mechanical strengths (denoted as σ_x and σ_z, respectively), it is possible to notice a greater convergence in the experimental values of rubberized compounds than CTR one. The rubber-based functionalization reduces the typical mechanical anisotropy of the 3D printed samples, as a result of the great structural compaction and inter-layer adhesion discussed in 3.1 section. The percentage deviation between the σ-values in Z-direction and X-direction was 43% for CTR and 25%, 12.5%, and 41% for RP100, RP50-RG50, and RP25-RG75, respectively. Reducing the post-hardening directional behaviour of 3D-printable cementitious materials is one of the most felt challenges in the field of AM technologies. In terms of design optimization for civil and architectural applications, enhancing mechanical isotropy has positive implications regarding the possibility of exploiting the full potential of digital fabrication. The observed divergences in σ-values measured in different loading directions can be attributable to two aspects: a) the orientation between applied stress and printed filaments; b) the direction-dependent compaction of the extruded material. From Table 3, σ_x demonstrates the best performance under compression, presumably due to the favorable orientation of the filaments for the applied pressure, which allows them to act as a series of columns with high compressive resistance, minimizing the mechanical stress at the weak inter-filament interfaces. A further reflection concerns the non-homogeneous densification that can be detected in a printed structure. The pressure exerted during the extrusion process through the pumping system is higher in the longitudinal direction (extrusion direction) than...
the other orientations, maximizing the compaction of the material. Conversely, in the perpendicular direction (Z-direction) the compaction is mainly related to the self-weight of the mortar. Similar remarks can be found in [40, 41].

To appreciate the effect of tire rubber aggregates on the elasto-mechanical properties of the printable mortars, the flexural stress-strain diagram is presented in Figure 12, considering the most representative specimens of each formulation. The addition of tire rubber aggregates significantly changes the deformation capacity of the mortars. In RCMs occurs an average E-value decrease of 95% compared to the neat sample, due to the lower elastic modulus of rubber than sand. For the same rubber-sand replacement levels, there are no significant variations of E with the tire particle size. The reduced modulus improves the deflection properties, prevents the brittle failure of the material, and leads to potential improvement in vibro-acoustic damping [42-43]. The effect of particle size is well evident on the ε-properties. The higher the RG content, the higher the material ductility. Reda Taha et al. [44] explained this behaviour by the ability of rubber inclusions to absorb part of the energy the matrix is subjected to, and therefore the rubber-cement material can absorb more mechanical energy before fracturing compared to the neat matrix. Polymer inclusions acted as obstacles to crack initiation and propagation that could have had a preferential path through the mineral aggregates. Therefore, during propagation, cracks must overcome the rubber-cement bond, circumnavigating the polymer particle. This prolongs the propagation path, postponing the failure process in the material [45]. In this context, the coarse fraction has a much more efficient effect on crack resistance than the fine one, since the larger particle size implies a longer propagation path for cracks inside the matrix, enhancing the material deformability and, as previously mentioned, the mechanical strength.

![Figure 12: Load-strain diagram resulting from four-point flexural test](image)

### 3.4. SIL Analysis

Figure 13 shows the SIL vs. f experimental spectra, recorded in low-middle-frequency (Figure 11a) and high-frequency (Figure 11b) bands.

Generally, for best acoustic insulation performance, a barrier material should meet specific requirements: smooth and high mass per unit area, non-porous structure, and a certain stiffness to enhance the reflection of noise wave, effectively encapsulate the incident sound, and minimize vibratory phenomena, respectively [46]. Following the characteristics listed above, it would be intuitive to deduce that the plain mortar can provide better sound-insulating behaviour than rubberized compounds. Although the modification with lightweight rubber aggregates reduces the unit weight and stiffness of the material, optimal acoustic properties are preserved. According to sound transmission theory, when the material damping is increased due to the addition of elastomeric elements, the vibro-acoustic insulation is also enhanced [47]. Besides that, additional advantages related to the versatility of lightweight cementitious materials are added: structural dead-weight reduction, thermal and fire resistance, high flexibility in fabrication and handling [48]. Globally, better attenuation performances are recorded in the high-frequency range (> 500 Hz), where the low penetrating power of the acoustic waves promotes the reflection effect of the material. Indeed, a maximum SIL-value of 21 dB was reached for the high-frequency region, while 13 dB peak-value occurred at low-middle frequency.
In relation to the rubber particle size, the coarse polymer aggregates (RG), having greater inertia than fine ones, tend to dissipate higher acoustic energy rates when subjected to vibration, promoting the insulating behaviour of the material. This may explain the best acoustic performance of the RP50-RG50 in the low-middle-frequency region. However, the positive effect of RG is less in the RP25-RG75 mix, where worse acoustic properties are observed. An explanation for this evidence can be found in the porous interface between cement and RG. The increase of the percentage RG increments the presence of interfacial voids, which represent weak points for sound attenuation [49]. Above 2000 Hz, RP100 exhibits slightly superior insulation properties compared to other samples, probably due to the functional matching between microstructural homogeneity, related to RP-cement cohesion, and acoustic shielding properties.

4. Conclusions

In this research work, the microstructural, mechanical, and acoustic properties of 3D printable cement mortars modified with tire rubber aggregates deriving from ELTs were investigated. The grain size of rubber particles is a key factor in modulating the physical-mechanical behavior of the material. By considering the same sand-rubber replacement level, fine polymer fractions (RP) are preferred to obtain better adhesion with the cement matrix and greater bulk density reduction, which is related to better performance in terms of lightweight and high-frequency acoustic insulation. Conversely, coarse rubber aggregates (RG) provide a positive effect considering the lower decrease in mechanical strength, good increase in ductility, and notable sound attenuation properties at low-middle
frequency. However, their content must be correctly balanced to avoid self-defeating effects, due to the poor cohesion with the cement paste. Concerning the microstructure, an overall reduction in porosity and refinement in pore size distribution were detected in the rubberized compounds than CTR mix, implying valuable potential properties in terms of durability and thermo-acoustic performances. Regardless of the particle size, the rubber modification improves the print quality of the mixes over the plain mortar in terms of inter-layer adhesion, promoting the mechanical isotropy of the hardened material. This effect is a direct consequence of the change in rheology that rubber induces in the cementitious mixes: increasing in fresh material fluidity and appropriate conditions of surface humidity between the printed filaments. The information derived from this investigation can be potentially useful to tire recycling industry in the optimized production of rubber particles used like recycled aggregates in cementitious mixes.

Future implementations of this research will be focused on two topics:

- Improvement of rubber-cement adhesion properties to enhance the mechanical strength of the rubberized compounds. In this framework, several chemical-physical pre-treatments of rubber waste were proposed to increase the compatibilization between rubber aggregates and the cement paste [50]. It will be necessary to consider numerous aspects such as sustainability of the approach, efficiency, and compatibility with additive manufacturing technology.

- Investigate the potential technological applications of 3D printable rubber-cement mixes. The results of the experimentation conducted by the authors suggest non-structural civil and architectural applications where strength is not a priority but greater lightweight, deformability, durability and thermo-acoustic insulation properties are required. Pre-cast members including insulating bricks in masonry, noise-reducing pavements, or acoustic barriers are possible applicability ways, where the digital fabrication could bring technological benefits in terms of design freedom (production of complex functional shapes, aesthetics, and components assembly) and selective use of materials in the manufacturing.

5. Declarations

5.1. Author Contributions

Conceptualization, M.V. and M.S.; methodology, M.S.; data curation, M.V. and M.S.; writing—original draft preparation, M.S.; writing—review and editing, M.V. and M.S.; supervision, M.V. All authors have read and agreed to the published version of the manuscript.

5.2. Data Availability Statement

The data presented in this study are available in article.

5.3. Funding

This research was performed thanks to the Sapienza University direct financing for PhD student Matteo Sambucci called “Avvio alla Ricerca”. Title is: “Study and optimization of rubber-concrete additives with recycled rubber that can be used through additive manufacturing: Optimization of thermo-acoustic, rheological and mechanical properties”.

5.4. Acknowledgements

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5.5. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Susceptibility Assessment of Single Gully Debris Flow Based on AHP and Extension Method

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Abstract

Debris flow mainly happens in mountainous areas all around the world with deadly social and economic impacts. With the speedy development of the mountainous economy, the debris flow susceptibility evaluation in the mountainous areas is of crucial importance for the safety of mountainous life and economy. Yunnan province of China is one of the worst hitting areas by debris flow in the world. In this paper, debris flow susceptibility assessment of Datong and Taicun gully near the first bend of Jinsha River has been done with the help of site investigation and GIS and remote sensing techniques. Eight causative factors, including slope, topographic wetness index, sediments transport index, ground roughness, basin area, bending coefficient, source material, and normalised difference vegetation index, have been selected for debris flow susceptibility evaluation. Analytical hierarchy process combined with Extension method has been used to calculate the susceptibility level of Datong and Taicun gullies. The evaluation result shows that both the gullies have a moderate susceptibility to debris flow. The result suggests that all the ongoing engineering projects such as mining and road construction work should be done with all precautionary measures, and the excavated material should adequately store in the gullies.

Keywords: Geological Disaster; Debris Flow Susceptibility; GIS and Remote Sensing; AHP; Extension Method.

1. Introduction

Debris flow is a two-phase fluid composed of soil, rock or organic matter flowing downslope under the influence of gravity. Debris flow can be very rapid and it may happen without any warning, because of this nature debris flow is one of the most hazardous geological disaster. Debris flow usually happens in mountainous areas. With the rapid development of the mountainous economy, the demand for soil resources increasing, due to the lack of resource disorders, the circulation zones and accumulated areas generated by debris flow activity is generally comparatively flat, frequently becoming an important place for mountain villages and towns to build, mountain rail, highways, and other transport hubs often choose the large and medium debris flow accumulated areas. The world's population is estimated to live nearly 10 percent in mountainous areas [1]. Natural disasters in mountainous areas, represented by landslide and debris flow, pose a great threat to human life and property [2-5]. The debris flow is a devastating geomorphological event due to high velocity (up to 56 km per hour) and good erosion quality besides the statistics; it can be combined
with large boulders and other rock remains [6]. Debris flow can move the object as big as a building or fill up structures with the fast accumulated deposits and organic material [7, 8].

In recent decades the study of the susceptibility to debris flow is rapidly improving with the latest growth of 3S and computer technology [9]. The GIS techniques make the debris flow susceptibility comparatively relaxed and these tools are very useful in data assessment. In general, the different approaches used in determining susceptibility to debris flow can be divided into qualitative and quantitative methods [10]. In the literature, many researchers worldwide worked on the susceptibility assessment of debris flow and applied different research methodologies, including artificial neural network [11], analytic hierarchy process [12-14], frequency ratio (FR) [15, 16], weights of evidence [17], certainty factor [18], factor analysis method [19], logistic regression [20, 21], gradient boosting machine [22], bivariate and multivariate statistical analyses [23], Fuzzy c-means clustering [24, 25], index of entropy [26], Extension method [13] and information value method [27].

Despite significant scientific advancements in debris flow assessment, only a few studies are available on the susceptibility assessment of a single-gully debris flow in the literature. In consideration of the shortcomings of previous studies, the foremost goal of this article is to study the susceptibility assessment of a single-gully debris flow. The debris flow susceptibility assessment of Datong and Taicun gully, sharing their accumulated fan, has been considered a research object in this study. This assessment can help prevent the consequences of future events and understand the importance of the main factors that cause debris flows. Eight evaluating factors including, slope, Topographic Wetness Index (TWI), Sediments Transport Index (STI), ground roughness, basin area, bending coefficient, source material and Normalised Difference Vegetation Index (NDVI) were selected for debris flow susceptibility assessment on the basis of field investigation, 3S technology and previous research practices.

To conclude how key variables influence debris flow susceptibility, the numerical weights of the individual factors according to their influencing power in debris flow susceptibility has been determined with Analytical hierarchy process (AHP). The AHP method has been widely used in hazard evaluation [12, 28-30]. One of the most powerful characteristics of the AHP method is its potential to determine quantitative and qualitative criteria and alternatives on the same scale of preferences [31]. In addition, weights and major variables have been combined with the Extension method to evaluate debris flow susceptibility level of the debris flow gullies.

The key objectives of this study are; to establish key parameters of single-gully debris flow using field investigation and 3S technologies, to quantify the impact of key parameters on debris flow susceptibility, and propose a combined model that explains debris flow susceptibility level. The AHP method, when combined with Extension theory, was found to determine the susceptibility of single-gully debris flow successfully. The statistical ability and accuracy of the findings were found to be accurate by comparing with the previous studies and the ground conditions of debris flow gullies. The findings acquired in this research have practical consequences for the implementation of potential debris-flow disaster prevention and hazard reduction measures in similar single gully debris flow catchments.

2. Study Area

The research area is located near the first bend of the Jinsha River, which is connected to Shigu Town, Yunnan Province, China. Datong and Taicun gully located on the right bank of the Jinsha River at a distance of 4.4 km towards the north from the first bend of the Jinsha River. The study area is positioned in the area of intense collision, extrusion, and compression between the Indian and Eurasian plate, forming the contraction and sliding of different block tectonic units. Under the comprehensive action of the endogenic and exogenic geological process, the present deep valley and high mountain alternate arrangement are finally formed. The geographical location of the study area is shown in Figure 1.

The elevation difference in the study area is generally above a kilometer; the lowest elevation is along the Jinsha River, with an altitude of 1850 m. The highest elevation is 5596 m near the Yulong Snow Mountain, and the elevation of the debris flow catchments is mainly 1800~3200 m (Figure 1).

2.1. Topography

According to the geographical zone of China, the study area belongs to the southwestern Tibetan Plateau geomorphic region. Further detailed division, it belongs to the freezing denudation and erosion cutting plateau area in western Sichuan-southern Tibet to the freezing denudation and erosion cutting sub-region in southern Qinghai-western Sichuan. Divided by genetic type, the study area is dominated by erosion, denudation, and ice erosion. Divided by morphological characteristics, the first bend of the Jinsha River is a typical alpine canyon area. The river is deeply formed a "V"-shaped canyon. The average width of the river is usually 80~150 m. However, the research area belongs to the wide valley section, with the river width of about 300~500 m.
The study area has typical alpine valley geomorphology. The slope angle of Datong and Taicun main gullies is greater than 35° with some up to 50-65° and more (Figure 2). The topographic features of the Datong and Taicun gully are shown in Table 1.

Table 1. Topographic features of Datong and Taicun Gully

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Datong gully</th>
<th>Taicun gully</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basin area /km</td>
<td>4.39</td>
<td>4.63</td>
</tr>
<tr>
<td>The linear length of the main gully /km</td>
<td>2.69</td>
<td>2.25</td>
</tr>
<tr>
<td>Main gully bending coefficient</td>
<td>1.02</td>
<td>1.03</td>
</tr>
<tr>
<td>The maximum relative elevation difference /km</td>
<td>1.144</td>
<td>1.339</td>
</tr>
<tr>
<td>The main gully curve length /km</td>
<td>2.76</td>
<td>2.31</td>
</tr>
<tr>
<td>The average ratio of the main gully /%</td>
<td>22.75</td>
<td>22.53</td>
</tr>
</tbody>
</table>

Figure 1. Geographical location of the study area

Figure 2. Slope map of Datong and Taicun gully

2.2. Climate

The monsoon climate characteristics in the Jinsha River basin are characterized based on the dry and wet seasons affected by the southwest and southeast monsoon. The precipitation is concentrated on the rainy season in the watershed, so the heavy rains in the watershed mainly occurred from June to August. The average annual and monthly rainfall of the study area is 834.3 and 69.7 mm, 24 h maximum rainfall was 106 mm, and the average number of precipitation a year is 138 days. The average temperature of the study area is 10.74°C, the extreme maximum and minimum temperature is 32.3 and -10.3°C (Figure 3). The average wind speed of 4.83 m/s, with a maximum wind speed of 18.3m/s, the maximum wind direction is W.
2.3. Geological and Engineering Geological Settings

The exposed geological strata of Datong and Taicun catchments consist of Quaternary colluvium deposits (Qk), Crystalline Limestone and marbles (C1), and Dolomite or limestone (D1h). The main exposed strata in the study area are Cambrian, Devonian, and Triassic; the geological map of the study area is shown in Figure 4. Since the Paleozoic, the study area has undergone multi-stage movement and transformation, forming more complex folds and faults with different properties. Due to the strong extrusion of the Indian plate into the Eurasian plate, large-scale superimposition, dislocation, and slip occurred among the blocks divided by the fault zones, resulting in large-scale thrust nappe and translational shear or strike-slip in the region. The study area belongs to east mountain fault zone of the Jinsha River, and the geological structural activity is relatively strong. The Daju-Lijiang fault and Longpan-Qiaohou fault are active that highly impact the study area. According to the relevant seismic data several earthquakes of magnitude ≥4.7 has been recorded in the study area as shown in the Figure 5(a). The seismic activity in the study area is strong and the seismic intensity reaches level IX (Figure 5(b)).

Figure 4. Geological map of the study area
The volume of loose material is an essential parameter for the debris flow initiation. The distributed loose source material was calculated during the field investigation. The distributed landslide collapse, rock avalanche, debris flow deposits, and unstable slopes were found in the study area. The accumulated loose source material varies from a minimum of 1mm to a maximum of 1 m in size. Due to the intrusion of limestone in the watershed, stone quarries existing in both the debris flow gullies; producing large amounts of loose material (Figures 8 and 12). Distributed loose course material and medium to large size crystalline limestone and marble boulders were found buried in loess in the accumulated fan. The viscosity of the debris flow residue material is normally determined by the nature of available material resources. Considering these, two parallel samples were collected from Datong gully accumulation fan during the field investigation for the sieve analysis test (Figure 6 (a, b)). The fine material of less than 1mm was taken back to the laboratory for laser particle analysis test. The analysis confirms that the Datong gully debris flow accumulation is mainly comprised of moderately textured soils, and the clay volume (<0.005 mm) is scarce (1.72%); consequently, the debris flow can be assumed to be low viscosity [32].

Based on the field investigation, the estimated source material statistics of Datong and Taicun gully are $151.53 \times 10^4$ and $15.98 \times 10^4 \text{ m}^3$. In which $2.18 \times 10^4 \text{ m}^3$ in Datong and $2.17 \times 10^4 \text{ m}^3$ in Taicun gully is unstable material and likely to be part of debris flow initiation. The characteristics of the Datong and Taicun watersheds are shown in Figures 7 to 13.

The groundwater also plays a major role in landslide and debris flow initiation. Groundwater flows in the pores and fractures of soil and rocks affect the mechanical and environmental boundary conditions such as weakening soil bond strength, reducing friction angles, and apply seepage forces [33, 34]. There are three types of groundwater present in the study area, i.e., underground pore water in loose deposits, bedrock fissure water in hillsides, and karst phenomena are developed to different degrees in these two gullies, so there is a small amount of karst water.

Figure 6. The particle size distribution of the two parallel sample in Datong gully (a) sample 1, (b) sample 2
Figure 7. Full view of Datong and Taicun gully from the accumulated fan

Figure 8. Datong gully (a) Stone quarry, (b) Crushed boulder deposits, (c) loose gravel and boulder deposits
Figure 9. The boundary between the ancient debris flow and river facies

Figure 10. Taicun gully (a) caving area at main channel left side, (b) loose gravel and boulders at the bottom of the main channel

Figure 11. Overview of the artificially excavated slope profile for a house construction near highways
3. Material and Methods

3.1. Data Used

This study includes four main stages; (I) the first stage dealt with data acquisition and information about the study area. These data and information include geotechnical, geological, hydrogeological, topographical, and rainfall information. (II) The second stage dealt with selecting causative factors based on the field investigation and previous research experiences. (III) In the third stage, the weight and the influencing power of individual parameters in debris flow initiation have been calculated using the AHP method. (IV) Finally, the susceptibility level of Datong and Taicun gully was evaluated using the Extension method. The flowchart of the steps involved in the susceptibility evaluation of Datong and Taicun gully is given in Figure 14.
Several causative factors influence the susceptibility of debris flow. The Multi-Criteria Analysis (MCA) with the concept of weight has been used for debris flow susceptibility zonation of Datong and Taicun gully. MCA in the paper context is also referred to as MADA (Multi-Attribute Decision Analysis), wherein a number of causative factors are analysed to reach a common objective, i.e., debris flow susceptibility assessment. In the present studies, AHP combined with Extension theory has been used for the Datong and Taicun gully debris flow susceptibility assessment.

3.2.1. Analytical Hierarchy Process

One of the primary concerns in decision theory or multi-parameter evaluation is estimating the relative weight of each factor and its influence; in our case, debris flows susceptibility with respect to the other. This is a task that involves human judgment complemented by mathematical methods. As all conditional factors cannot be weighted equally for the susceptibility assessment, a weighted technique must be used where the relative value of the parameters defines the weightage.

There are a variety of approaches available to deal with such concerns. In the present study, we used Saaty's analytic hierarchy process [35], referred to as AHP, the most broadly approved scaling method. The weights of a variable establish a pair-wise judgemental matrix of variables whose entries signify the strength with which one component is dominant over another. AHP is a process based on decision theory in which it is necessary to compare each criterion from a set of choices or alternatives. It indicates the most accurate methodology for calculating the weight of criteria and estimating the relative magnitude of factors through pair-wise comparison with experts’ judgment and experience. It directs the importance of a certain factor in debris flow assessment by co-relating with other elements through a statistical comparison. The scores given are based on reasonable prioritisation of the factor for inducing susceptibility of debris flow and depend on the estimation of the expert following the evaluation scale given by Saaty (2008) [36]. The grading of comparative factors is done by allotting weight ranges between 1 and 9, where 1 directs the same importance, and 9 represents the extreme importance of a particular factor over another, as shown in Table 2. The AHP calculations in this study were done in Microsoft Excel using the following steps [37]:

(i) Add the quantities in the pair-wise matrix's columns.

\[ C_{ij} = \sum_{i=1}^{n} C_{ij} \]  

(ii) To obtain a normalised pair-wise matrix, the matrix parameter was divided by the column's sum, respectively.
\[ A_{ij} = \frac{C_{ij}}{\sum_{i=1}^{n} C_{ij}} \]  

(iii) To obtain each criteria weight, the summation of the matrix’s normalised columns was divided by the amount of parameters applied.

\[ W_{ij} = \frac{\sum_{j=1}^{n} A_{ij}}{n} \]

Where \( n \) is the number of parameters and \( C_{ij} \) is the pair-wise matrix, \( A_{ij} \) is the normalised value and \( W_{ij} \) is the criteria weight.

Table 2. Pair-wise comparison 9-point rating scale in AHP, after [35]

<table>
<thead>
<tr>
<th>Dominant values</th>
<th>Description</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Equal importance</td>
<td>Two factors contribute equally</td>
</tr>
<tr>
<td>3</td>
<td>Moderate importance</td>
<td>judgement slightly favour one factor over another</td>
</tr>
<tr>
<td>5</td>
<td>High prevalence</td>
<td>judgement highly favour one factor over another</td>
</tr>
<tr>
<td>7</td>
<td>Very high prevalence</td>
<td>Activity is very highly favoured over another</td>
</tr>
<tr>
<td>9</td>
<td>Extremely high prevalence</td>
<td>Activity is extremely favoured over another</td>
</tr>
<tr>
<td>2, 4, 6, 8</td>
<td>Intermediate values</td>
<td>used when comprises is needed</td>
</tr>
</tbody>
</table>

Though the comparisons in AHP are assigning by expert judgement, but still there could be inconsistency found in calculations. The consistency is derived in AHP by coherence ratio, which is given by Saaty (2008) [36] given in Table 3.

Table 3. Random index values

<table>
<thead>
<tr>
<th>n</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>RI</td>
<td>0.0</td>
<td>0.0</td>
<td>0.58</td>
<td>0.90</td>
<td>1.12</td>
<td>1.24</td>
<td>1.32</td>
<td>1.41</td>
<td>1.45</td>
<td>1.49</td>
</tr>
</tbody>
</table>

RI= random consistency index.

\[ CR = \frac{CI}{RI} \]  

Where;

\[ CI = \frac{\lambda_{max} - n}{n} \]

Where \( \lambda_{max} \) is the Principal Eigen Value and can be determined as; \( \lambda_{max} = \Sigma \) of the products between each variable of the priority vector multiplied by column totals. Whereas \( n \) is the total number causing parameters.

The consistency ratio will be calculated in order to find the continuity of the pair-wise compared weights [38]. The uniqueness of the AHP method is that it provides \( CR \) as a relationship between the degree of consistency and inconsistency. The suitable value of \( CR \) is 0.1 for all large matrices, i.e., \( n>5 \). Therefore, a \( CR \) of 0.1 or less is of reliable importance [39]; however, a \( CR \) above 0.1 needs to review the conclusions in the matrix.

3.2.2. Extension Theory

Cai (1983) proposed a new transverse discipline, Extenics [40]. Extenics is an interdisciplinary subject; the basic method is the Extension method. The extensibility of things, laws and methods can be studied by formalized models, which can ultimately solve the contradictions. A comprehensive evaluation of things using the knowledge of extension requires an effective combination of the quality and quantity of the evaluation object’s characteristics.

The impacts of various causative variables on the susceptibility of debris flow can vary considerably. Influence deviation can be seen as an issue of contradiction and inconsistency. The method of extension is designed to solve this sort of difficulty by integrating the various degrees of influence of all causative variables in order to deduce an ultimate and complete susceptibility result [13].

Extension set and matter-element theory are the fundamental theories of the Extension method. The matter-element treats things as a ternary group \( R \). The matter-element can be represented by Equation 6:

\[ R = (N, C, V) \]
In which the basic element (debris flow gully) can be represented by \( N \), the basic element’s characteristics (Causative factors) can be represented by \( C \), and the value of the characteristics (for susceptibility analysis) can be represented by \( V \) respectively.

In the classical fuzzy set, the value range is \([0, 1]\), in which 0 denotes that things have certain characteristics, 1 denotes that things do not have certain properties. In simple words, 0 and 1 denote whether things have certain characteristics or not, respectively. Compare to the classic fuzzy set \([0, 1]\); Extension sets are represented by real numbers \([-\infty, +\infty]\); which means that the extension theory not only study whether a component belongs to a set but also defines the grade of its belonging [41, 42].

Let \( U \) be the environment of an item and \( x \) be a component of \( U \), the extension set of \( X \) on \( U \) is described as the set of ordered pairs as shown in Equation 7:

\[
X = \left\{ \frac{x, y}{x} \in U, y = k(x) \in (-\infty, +\infty) \right\}
\]  
(7)

In which \( k(x) \) is the correlation function of the \( X \) extension set, which is used to describe the connection between the \( x \) variable and the \( X \) extension set. The consequence of \( k(x) \) may be positive, negative, or zero. When \( k(x) > 0 \), \( X \) is referred to as a positive class, simply mean belongs to the set and defines the grade of its belonging to the set. When \( k(x) < 0 \), \( X \) is considered a negative class simply means that it does not relate to the set, it defines the grade to which the parameter does not belong to the set. When \( k(x) = 0 \), \( X \) is considered a boundary zero [43].

3.2.2.1. Extension Evaluation Steps

(a) Determining Extension set:

\[
R_{ij} = (N_j, C_i, V_{ij}) = \left[ \begin{array}{c}
N_j, C_1, V_{1ij} \\
C_2, V_{2ij} \\
\vdots, \ldots \\
C_n, V_{nij}
\end{array} \right] = \left[ \begin{array}{c}
N_j, C_1, (a_{11j}, b_{1j}) \\
C_2, (a_{21j}, b_{2j}) \\
\vdots, \ldots \\
C_n, (a_{nj}, b_{nj})
\end{array} \right]
\]  
(8)

Where \( N_j \) represents the debris flow susceptibility level, \( C_i \) (i=1, 2, 3,...,n) represents the parameters of debris flow susceptibility, \( V_{ij} \) represents the range of parameter’s values, and the classical domain is the value of each parameter in different evaluation levels.

Nodal elements are ranges of values for each susceptibility level for each factor:

\[
R_p = (P, C, V_p) = \left[ \begin{array}{c}
P, C_1, V_{1p} \\
C_2, V_{2p} \\
\vdots, \ldots \\
C_n, V_{np}
\end{array} \right] = \left[ \begin{array}{c}
P, C_1, (a_{1p}, b_{1p}) \\
C_2, (a_{2p}, b_{2p}) \\
\vdots, \ldots \\
C_n, (a_{np}, b_{np})
\end{array} \right]
\]  
(9)

Where \( P \) denotes debris flow gully in our case, that is, the whole level of debris flow susceptibility, and \( V_p \) denotes the range of values of \( P \) with respect to the factor \( C \), that is, the influencing factor of \( P \).

(b) Determining the Matter-element:

\[
R = (P, C, v) = \left[ \begin{array}{c}
P, C_1, v_1 \\
C_2, v_2 \\
\vdots, \ldots \\
C_n, v_n
\end{array} \right]
\]  
(10)

Where, \( P \) is the debris flow gully to be assessed, \( C_i \) is the factor influencing the susceptibility level, and \( v_i \) represents \( P \)’s level of \( C \), which is the data collected from the thing to be evaluated.

(c) Determining the value of the correlation function

The correlation degree of each individual evaluation index is:
Recent advances in GIS software and increased computational ability make it conceivable to use a considerably large amount of independent variables in data-driven debris flow susceptibility assessment. Debris flows are induced by various external ecological and internal geological variables [44]. Normally, debris flow formation involves three constraints: loose source material, topography, and rainfall [12]. The loose source material is the physical source of debris flow events and is linked to section lithology and geological structures by influencing the conflux mechanism [45]. Table 4 referenced the excessively used causative factors by various researchers for debris flow susceptibility assessment.

Precipitation is an important parameter that impacts debris flow susceptibility. However, in the current study, two debris flow gullies have been studied located in the same area receive the same amount of precipitation; therefore, precipitation is not considered as a debris flow causative parameter. The geographical distribution of soil moisture and sub-surface water pressure is influenced by topography, which is a major element in the spatial variability of hydrological environments [46, 47]. Because debris flows contain large quantities of water, the hydrological-topographic variable of soil moisture has been regarded as an essential parameter for debris flow evaluation in this study. Amongst several variables of the hydrological factors, topographic wetness index [48], sediments transport index [49], and ground roughness [50] have been selected in this work to examine the impact of these variables on debris flow susceptibility. The STI specifies the soil erosion, TWI assess soil moisture spatial distribution and ground roughness directly influence the ability of ground confluence and seepage. Hence TWI, STI, and ground roughness were taken into consideration in this article.

The average elevation of the Tibetan plateau is about 4500m; the Shigu area belongs to the low elevated area of the Tibetan plateau; therefore, the elevation is not considered in this study.

The amount of source material volume represents lithological properties and fault distribution to some degree; therefore, lithology and fault distribution were not considered. The basin area and bending coefficient of the main channel also represent the amount of flood intensity and its effect on the lithological composition of the debris flow catchment. Therefore basin area and bending coefficient are also taken as causative factors in this study.

As previously stated, causative factors can be used as driving factors in forecasting potential outbreaks of debris flows in studied areas [51]; however, there is no specific principle for choosing these factors exist [52]. In the present study, the causative parameters were chosen amongst those widely reported in the literature for debris flow susceptibility assessment. Field investigation and spatial analysis were carried out to determine the influence of each of these variables on the debris flow distribution in our study area. The correlation between debris flow distribution and different indicators proposed a causal influence. The eight most relevant influencing factors were preferred in this study, including slope (F1), TWI (F2), STI (F3), ground roughness (F4), basin area (F5), bending coefficient of the main channel (F6), source material (F7), and NDVI (F8) were selected as evaluation indicators in this study. These variables directly or indirectly impact the happening of debris flows and have been widely used in the assessment of debris flow susceptibility [45, 53].

F1 represents the debris flow gully slope angle. The slope angle was generated from the study area DEM with the help of ArcGIS software. Slope stability have direct physical relationship with the debris flow initiation and the phenomenology of landslides; greater angle indicated greater slope instability, and vice versa. The slope of the debris flow has a response angle of 20°- 30°, the maximum angle at which the loose material is stable [61].

\[
K_i(v_{ij}) = \begin{cases} 
-\frac{\rho(v_i, V_{ij})}{|V_{ij}|} & v_i \in V_{ij} \\
\frac{\rho(v_i, V_{ij})}{\rho(v_i, V_{pi}) - \rho(v_i, V_{ij})} & v_i \notin V_{ij}
\end{cases}
\]

\[
\rho(v_i, V_{ij}) = |v_i - a_{ij} + b_{ij}| - \frac{b_{ij} - a_{ij}}{2} \quad |V_{ij}| = |b_{ij} - a_{ij}|
\]

\[
\rho(v_i, V_{pi}) = |v_i - a_{pi} + b_{pi}| - \frac{b_{pi} - a_{pi}}{2}
\]

**4. Debris Flow Susceptibility Assessment**

**4.1. Assessment Factors**

The selection of causative variables is a requirement for pre-processing in debris flow susceptibility assessment. The eight most rel...
Table 4. Assessment variables frequently used by researchers in debris flow susceptibility assessment

<table>
<thead>
<tr>
<th>Factors</th>
<th>[54]</th>
<th>[55]</th>
<th>[56]</th>
<th>[12]</th>
<th>[57]</th>
<th>[58]</th>
<th>[23]</th>
<th>[19]</th>
<th>[45]</th>
<th>[59]</th>
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</tr>
<tr>
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<td>✓</td>
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<td>✓</td>
</tr>
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<td>✓</td>
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<td>✓</td>
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<td>✓</td>
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<td>Main channel length</td>
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<td>✓</td>
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<td>✓</td>
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<tr>
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<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

F2 represents TWI value of the debris flow gullies. Beven and Kirkby first proposed the topographic wetness index [48]. Many researchers in the literature used the TWI is a causative factor for landslide and debris flow susceptibility assessment [58, 62, 63]. The areas of high TWI values provide more favourable conditions for landslide and debris flow initiation [62]. The TWI values of the Datong and Taicun gully was determined from DEM by means of the spatial analysis tool in ArcGIS 10.3 using Equation 14:

$$TWI = \ln (F_{ac} - 0.001)\left(\% \text{slope/100} + 0.001\right)$$

Where $F_{ac}$ is Flow accumulation which could be acquired from DEM file.

F3 represents STI value of the debris flow catchments. Sediments transport index which originates from the Universal Soil Loss Equation (USLE) [64] is the measure of soil erosion and carrying in the flow channel [65]. It is the volume of material transport ability of a flow along the water channel. The larger the slope length, the higher the soil erosion because of the water deposition at the bottom. The STI values were prepared from the DEM file with the help of ArcGIS 10.3 spatial analysis tool using Equation 15:

$$STI = \text{power}\left(\frac{F_{ac}}{22.13,0.6}\right) \times \text{power}\left(\sin\left(\frac{\text{Atan}\left(\frac{\text{slope}(\%)}{100}\right)}{0.0896,1.3}\right)\right)$$

Where $F_{ac}$ is flow accumulation.

F4 represents ground roughness; ground roughness also influences debris flows and is perceived to be a significant evaluation factor. The ground roughness indicates the surface area ratio to the projective area for a given area [12]. It directly influences the ability of ground confluence and seepage. Ground roughness of the study area was evaluated from DEM using spatial analysis tool in ArcGIS.

F5 represents the basin area of the debris flow gullies. Basin area has a direct impact on the quantity of the loose source material. In general, a broader basin area would lead to a greater amount source material [25]. The extent of the debris flows is determined by source material volume. As a consequence, basin area is considered a major influence factor in the debris flow susceptibility evaluation. It was obtained through ArcGIS and google earth.

F6 denotes bending coefficient of the debris flow gully main channel. Bending coefficient is the proportion of the main channel's curve length to conventional length. This proportion represents the debris flow discharge environment. Currents may strike and erode the exposed lithology of the curvy channel. The hillsides stability will be reduced, and enhance the amount source materials in the main channel [66]. A debris flow catchment with a small amount of loose material is often distinguished by an outward and straight channel; because the stacked source material will be washed with the established debris flow in the main channel [13].

F7 is the available source materials in debris flow catchments. Sufficient loose source material in the debris flow gullies are the primary conditions for the formation of debris flow. Once the debris flow source is activated, it will
pose the first impact to the loose material distributed in the formation and circulation areas of the debris flow gully. The investigation of solid source material accumulated in the debris flow gullies has a crucial impact on debris flow initiation. The source material data was collected and evaluated from the field investigation.

F8 denotes NDVI values of the debris flow gullies. The NDVI value describes the vegetation cover of the study area. The literature shows that most debris flows occur in areas with less vegetation cover and NDVI values, especially less than 0. The NDVI values varied from −1 to 1; the higher the NDVI score, the heavier the vegetation cover [11]. The high NDVI value decreases the runoff erosion of the gully and reduces the chance and potential of debris flow. The area without vegetation cover has rapid and favourable convection to landslide and debris flow. The NDVI value of the research area was evaluated from Landsat-8 imagery using ArcGIS Raster calculator.

4.2. Modelling Approach

Selecting the causing factors has highlighted a number of key parameters that are potentially important in the derivation of susceptibility assessment of debris flow. It is extremely necessary to combine them to drive a single representative value for susceptibility analysis; otherwise, they are independent variables which offer separate indication.

4.3. Assigning Weight

In this study, the AHP method was used to calculate the weight of one causative factor over another. One of the major benefits of using AHP is rearranging the complication of data set by the hierarchy with a pair-by-pair comparison between two variables, therefore minimising weighting inaccuracy while ensuring that different data processing is consistent. However, this method can vary from one expert to another, based on expert opinion, judgment, and ranking of the causative factor, which is, therefore, a minor drawback. Many researchers globally used the AHP method in their studies to assess debris flow susceptibility.

A pair-wise matrix was built, in which each criterion was correlated with other criteria, according to its importance, on a scale from 1 to 9, as can be seen in Table 4. As we have eight parameters, then we have eight by eight matrices. The diagonal matrix elements are always 1, and one needs to complete up the upper triangular matrix. If the value of the decision is on the left side of 1, we shall set the numerical value of the decision. Suppose the value of the decision is on the right side of 1. In that case, we shall set the inverse value to build a debris flow susceptibility model and provide a method to determine the variable weights in the linear debris susceptibility model. The importance matrices for eight variables have been created in Microsoft Excel [37]. An importance matrix table has been obtained by adjusting the value for a pair of two variables, as shown in Table 5. These relationship scores have been achieved by field investigation and previous research experiences. The individual criteria weight was obtained following the steps explained in Section 3.2.1. above.

<table>
<thead>
<tr>
<th>Table 5. Pair-wise comparison results of Debris flow causative factors</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Judgement matrix</strong></td>
</tr>
<tr>
<td><strong>F1</strong></td>
</tr>
<tr>
<td><strong>F2</strong></td>
</tr>
<tr>
<td><strong>F3</strong></td>
</tr>
<tr>
<td><strong>F4</strong></td>
</tr>
<tr>
<td><strong>F5</strong></td>
</tr>
<tr>
<td><strong>F6</strong></td>
</tr>
<tr>
<td><strong>F7</strong></td>
</tr>
<tr>
<td><strong>F8</strong></td>
</tr>
<tr>
<td><strong>(\Sigma (\text{Sum}))</strong></td>
</tr>
</tbody>
</table>

Note: F1 = slope, F2 = TWI, F3 = STI, F4 = ground roughness, F5 = basin area, F6 = bending coefficient, F7 = source material, F8 = NDVI

Let us look at how the pair-wise judgments were calculated with an understanding of F1 (slope). The significance matrix will be normalized and weighted using the "Eigen Vector" technique, as shown in Table 6.

<table>
<thead>
<tr>
<th>Table 6. Normalized value determination</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Comparison matrix</strong></td>
</tr>
<tr>
<td><strong>F1</strong></td>
</tr>
<tr>
<td><strong>(\Sigma (\text{Sum}))</strong></td>
</tr>
<tr>
<td><strong>Normalization</strong></td>
</tr>
</tbody>
</table>
The weight of the parameter F1 in debris flow susceptibility was calculated using Equation 3:
\[ \Sigma = 0.367+0.46+0.37+0.28+0.252+0.218+0.182=0.31 \] (weight of F1 (slope)) or 31%.

The weight of other parameters were calculated in the same way. The individual weight of each causative factor finally obtained as: F1 (0.31) > F2 (0.25) > F3 (0.17) > F4 (0.091) > F5 (0.078) > F6 (0.042) > F7 (0.032) > F8 (0.025) respectively.

The final stage is to calculate the consistency ratio to measure how consistent the judgement is. The consistency ratio was calculated with Equation 4.

In our case by calculation the CR = 0.08, which is less than 0.1; the ratio shows an acceptable degree of consistency in the pair-wise judgement, standing sufficient to distinguish the factor weights. The revision of the preferences matrix will be needed if the CR value is more than 0.1.

4.4. Susceptibility Assessment based on Extension Theory

Debris flow susceptibility are being calculated based on the causative variables and their weights. Four classes of susceptibility were established in this article: low, medium, high, very high. Since we have to analyse the susceptibility assessment of two debris flow gullies Datong and Taicun, in this paper, the matter-element was evaluated first for J=1 and 2, respectively. The eight causative factors, including slope, TWI, STI, ground roughness, basin area, bending coefficient, Source material and NDVI, are evaluated. The weight of each parameter calculated using the AHP method as shown in Table 5.

The variable values are given in Table 7. As per previous literature, the matter-element ranges [VL, VU] of the four susceptibility degrees, i.e. low, moderate, high and very high for each variable, are determined. The neighbourhood domain range can be derived either from previous practical practice or calculated from the maximum and minimum values of each variable in the field survey [13]. The second one is being used in this study. The ranges and the neighbourhood domains are given in Table 8. The correlation function can be determined using Equation 11. From the value of the correlation function, the degree of correlation of the evaluated parameters with respect to the grade t is obtained according to the Equation 16:

\[ K_t(P) = \sum_{i=1}^{n} W_i K_t(v_i) \]  (16)

Where \( W_i \) is the weight factor of each evaluation parameter, and \( \sum W_i = 1 \).

If \( K_t(P) = \max K_t(P) \), then the susceptibility of debris flow is ranked K.

<table>
<thead>
<tr>
<th>Gully name</th>
<th>F1 (°)</th>
<th>F2</th>
<th>F3</th>
<th>F4</th>
<th>F5 (km²)</th>
<th>F6</th>
<th>F7 (× 10⁴ m³)</th>
<th>F8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Datong gully</td>
<td>45</td>
<td>12</td>
<td>200</td>
<td>1.5</td>
<td>4.39</td>
<td>1.02</td>
<td>2.18</td>
<td>0.159</td>
</tr>
<tr>
<td>Taicun gully</td>
<td>43</td>
<td>12</td>
<td>174</td>
<td>2</td>
<td>4.63</td>
<td>1.03</td>
<td>2.17</td>
<td>0.13</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Susceptibility level and neighbourhood domain</th>
<th>F1(°)</th>
<th>F2</th>
<th>F3</th>
<th>F4</th>
<th>F5 (km²)</th>
<th>F6</th>
<th>F7 (× 10⁴ m³)</th>
<th>F8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>≤ 20</td>
<td>≤ 5</td>
<td>≤ 100</td>
<td>≤ 1.26</td>
<td>≤ 0.5 or ≥ 50</td>
<td>≤ 1.10</td>
<td>≤ 1</td>
<td>≥ 0.4</td>
</tr>
<tr>
<td>Moderate</td>
<td>20 - 30</td>
<td>5 - 10</td>
<td>100 - 200</td>
<td>1.26 - 1.46</td>
<td>0.5 - 10</td>
<td>1.10 - 1.25</td>
<td>1 - 10</td>
<td>0.24 - 0.4</td>
</tr>
<tr>
<td>High</td>
<td>30 - 40</td>
<td>10 - 15</td>
<td>200 - 300</td>
<td>1.46 - 2</td>
<td>10 - 30</td>
<td>1.25 - 1.40</td>
<td>10 - 100</td>
<td>0.16 - 0.24</td>
</tr>
<tr>
<td>Very High</td>
<td>≥ 40</td>
<td>≥ 15</td>
<td>≥ 300</td>
<td>≥ 2</td>
<td>≥ 30</td>
<td>≥ 1.40</td>
<td>≥ 100</td>
<td>≤ 0.16</td>
</tr>
<tr>
<td>neighborhood domain</td>
<td>0 - 70</td>
<td>0 - 25</td>
<td>0 - 500</td>
<td>0 - 3</td>
<td>0 - 40</td>
<td>0 - 2</td>
<td>0 - 200</td>
<td>-1 - 1</td>
</tr>
</tbody>
</table>

The correlation degree of each grade of eight causative parameters of the susceptibility degree evaluation model is calculated by using the correlation function equation for Datong and Taicun debris flow catchments respectively, as shown in Table 9.
Table 9. Correlation degree of individual parameter of Datong and Taicun gullies on debris flow susceptibility

<table>
<thead>
<tr>
<th>Level description</th>
<th>Datong gully</th>
<th>Taicun gully</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>F1</td>
<td>-0.157</td>
<td>-0.1178</td>
</tr>
<tr>
<td>F2</td>
<td>-0.0917</td>
<td>-0.0356</td>
</tr>
<tr>
<td>F3</td>
<td>-0.056</td>
<td>0</td>
</tr>
<tr>
<td>F4</td>
<td>-0.01255</td>
<td>-0.0024</td>
</tr>
<tr>
<td>F5</td>
<td>-0.0366</td>
<td>0.6068</td>
</tr>
<tr>
<td>F6</td>
<td>0.0037</td>
<td>-0.0032</td>
</tr>
<tr>
<td>F7</td>
<td>-0.01124</td>
<td>0.0378</td>
</tr>
<tr>
<td>F8</td>
<td>-0.0056</td>
<td>-0.0022</td>
</tr>
</tbody>
</table>

Note: F1 = slope, F2 = TWI, F3 = STI, F4 = ground roughness, F5 = basin area, F6 = bending coefficient, F7 = source material, F8 = NDVI

According to the combination weight coefficient and single correlation degree of eight evaluation factors, the comprehensive correlation degree of Datong and Taicun debris flow is calculated, as shown in Table 10. According to the evaluation results, the susceptibility level of Datong and Taicun debris flow is moderate. Datong and Taicun gullies are located at the same location, sharing their accumulation fan and because of the same environmental, topographic and geological environment, both catchments have similar debris flow susceptibility levels. The outcomes were compatible with the findings of the field investigation.

Table 10. Extension evaluation results of debris flow susceptibility of Datong and Taicun gully

<table>
<thead>
<tr>
<th>Gully name</th>
<th>Low</th>
<th>Moderate</th>
<th>High</th>
<th>Very High</th>
<th>Susceptibility Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Datong gully</td>
<td>-0.367</td>
<td>0.483</td>
<td>-0.198</td>
<td>-0.746</td>
<td>Moderate</td>
</tr>
<tr>
<td>Taicun gully</td>
<td>-0.376</td>
<td>0.536</td>
<td>-0.079</td>
<td>-0.776</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

To check the validation of our study, the results were compared with the work done by Li et al. (2017) [25] (Debris flow susceptibility in the Wudongde dam area, Jinsha River) and Liang et al. (2020) [24] (susceptibility assessment of debris flow based on a semi-quantitative method). These studies were selected for comparison because of their similar topographical, meteorological, and geological environment to our case studied (Datong and Taicun gully).

Li et al. (2017) [25] used eight causative factors to evaluate the susceptibility assessment of 22 debris flow catchments along the Jinsha River close to Wudongde Dam site, Yunnan province of China. Out of 22 debris flow gullies, Xiabatian and Zhugongdi catchments were taken on the basis of basin area and other topographic features to compare with our case studied (Datong and Taicun gullies). Li et al. (2017) [25] used the rock engineering system and fuzzy C-means algorithm (RES_FCM) for debris flow susceptibility assessment. Based on the evaluated results Xiabatian have high and Zhugongdi catchment has moderate susceptibility to debris flow. Xiabatian catchment shows high susceptibility to debris flow because of the available large amount of source material and higher bending coefficient values than our case study, as shown in Table 11.

Liang et al. (2020) [24] studied the susceptibility assessment of 21 debris flow catchments in pinggu district, Beijing using FA, FCM, and ECM to classify and evaluate the susceptibility level of debris flow. Based on the basin area and other topographic features, out of 21 catchments, the evaluation results of Tawa and Hundong gullies were compared with the evaluation results of the Datong and Taicun gullies. The evaluation results of Liang et al. (2020) [24] shows that Tawa has high and Hundong catchment have moderate susceptibility to debris flow. The basin area and the ratio of available source material in Tawa gully are higher than those of Datong and Taicun gullies, making
their susceptibility level high. In contrast, the Hundong gully has a comparatively small basin area and less source material; that’s why evaluation results show of moderate level.

From this comparison, it is worth noting that the basin area, bending coefficient, and the available amount of loose material in the debris flow catchments directly impact debris flow susceptibility level. The higher the amount of loose material higher will be the susceptibility level, even if the slope angle is than 30°. On the basis of the above comparison and field investigation analysis, we can conclude that the evaluation results of our study are satisfactory.

Table 11. Comparison of the current evaluation results to previous studies

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Datong gully</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Basin area (km²)</td>
<td>3.1</td>
<td>6.5</td>
<td>4.39</td>
</tr>
<tr>
<td>Slope (°)</td>
<td>36.1</td>
<td>41.8</td>
<td>45°</td>
</tr>
<tr>
<td>Bending coefficient</td>
<td>1.19</td>
<td>1.15</td>
<td>1.02</td>
</tr>
<tr>
<td>Vegetation cover</td>
<td>0.45</td>
<td>0.4</td>
<td>0.15</td>
</tr>
<tr>
<td>Source material (×10⁶ m³)</td>
<td>904</td>
<td>316</td>
<td>2.18</td>
</tr>
<tr>
<td>Susceptibility level</td>
<td>High</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
<tr>
<td>Taicun gully</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Basin area (km²)</td>
<td>6.65</td>
<td>18.457</td>
<td>6.443</td>
</tr>
<tr>
<td>Slope (°)</td>
<td>5.31</td>
<td>4.39</td>
<td>4.63</td>
</tr>
<tr>
<td>Bending coefficient</td>
<td>1.08</td>
<td>1.03</td>
<td>1.03</td>
</tr>
<tr>
<td>Vegetation cover</td>
<td>0.52</td>
<td>0.62</td>
<td>0.13</td>
</tr>
<tr>
<td>Source material (×10⁶ m³)</td>
<td>4.63</td>
<td>2.18</td>
<td>2.17</td>
</tr>
<tr>
<td>Susceptibility level</td>
<td>High</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

5. Conclusions and Recommendations

The research area is located in the northwest of Yunnan province, which belongs to the upper streams of Jinsha River Tibetan Plateau. The research area is tectonically active and causes a large amount of landslide and debris flow every year. The Datong and Taicun gullies are located near the first bend of the Jinsha River, which has a very important geological significance. The study area belongs to the east mountain fault Zone of the Jinsha River and the geological structure-activity is relatively strong. The presence of active geological faults and mining activities in the debris flow gullies make it more susceptible to landsliding and debris flow. The active mining work is producing a large amount of solid source materials of various sizes. A large amount of separated large-sized collapsed boulders scattered in the debris flow catchments, which is difficult to be carried in normal rain, but once the heavy rain occurs, these solid source materials from mining work and collapsed deposits can be part of debris flow initiation.

A detailed field investigation was organized to collect data regarding the susceptibility assessment of debris flow in Datong and Taicun gully. The collected field data was analysed and interpreted with the help of GIS and Remote sensing techniques. Eight causative factors, including slope, TWI, STI, ground roughness, basin area, bending coefficient, source material, and NDVI, were considered to investigate and calculate the susceptibility assessment debris flow of Datong and Tai gully using AHP and Extension methods. The weightage of each causative factor was evaluated with pair-wise comparison using the AHP and priority given based on expert views. The individual influencing weight of each causative factors finally obtained as: slope (0.31)>TWI (0.25)>STI (0.168)>ground roughness (0.091)>basin area (0.078)>bending coefficient (0.042)>source material (0.032)>NDVI (0.025) respectively. The susceptibility assessment of Datong and Taicun gully was done using Extension Theory, which has a good competency to solve inconsistency and contradiction complications.

According to the combined weight coefficient and single correlation degree of eight evaluation factors, the comprehensive correlation degree of Datong and Taicun debris flow is calculated. Based on the evaluation results, the susceptibility level of Datong and Taicun debris flow gully is moderate.

Datong and Taicun gully located at the same location, sharing their accumulation fan and because of the same environmental, topographic and geological environment, both the gullies results in similar susceptibility level. The method used in this study is simple and convenient to implement. The AHP method, when combined with Extension theory, was found to determine the susceptibility of single-gully debris flow successfully. The statistical ability and accuracy of the findings were found to be satisfactory by comparing the evaluation results with previous studies and field investigation judgements.

However, there are some limitations to the approach used in this study: (1) the precipitation, which is a key factor in determining debris flow susceptibility, was not taken into account in this study. Susceptibility findings would be more reliable if precipitation variance for debris flow catchments is used. (2) The instruments utilized during fieldwork are too simple, and some key parameters, such as the amount of source material per square kilometre, are not precise enough. Divergences between site investigation findings and indoor interpretation consequences could be reduced further. (3) In the AHP method, the weight is assigned to the causal parameters based on expert opinion and judgments. The weight could be different from the actual values. The identification of precise weights also necessitates unrelenting efforts.
Based on the debris flow susceptibility analysis of the Datong and Taicun gully, the following recommendations are suggested:

- All the construction work, especially road construction, which is undergone in the study area, should be constructed with all the precautionary measures and properly store the excavated material;
- All the waste material produced from the mining work should be removed from the gully or compacted layer-wise not to be carried easily by fluid in rainy seasons;
- For all kinds of engineering projects and villagers, safety measures should be taken into account;
- For the awareness, local inhabitant investment in counteractive steps on the debris flow is highly demonstrated in public education and early warning systems.

6. Declarations

6.1. Author Contributions

Q.M. was responsible for writing and graphic production of the manuscript; J.C. supervised the field investigation; W.Q. was responsible for the manuscript's revision; J.Y. and N. were responsible for the calculations part; G.R. and M.A. were responsible for proofreading and references. All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

The data presented in this study are available in article.

6.3. Funding

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6.4. Acknowledgements

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6.5. Conflicts of Interest

The authors declare no conflict of interest.

7. References

civil engineering journal
Performance of Ground Anchored Walls Subjected to Dynamic and Pseudo-Static Loading

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Abstract

This study investigates the response of pre-stressed anchored excavation walls under dynamic and pseudo-static loadings. A finite difference numerical model was developed using FLAC3D, and the results were successfully validated against full-scale experimental data. Analyses were performed on 10, and 20-m-height stabilized excavated slopes with 60° to 90° of inclination angle with the horizon to represent an applicable variety of wall geometries. In dynamic analysis, the statically stabilized models were subjected to 0.2 to 0.6g of the dynamic peak acceleration to evaluate the effect of ground acceleration on their performance. Furthermore, pseudo-static analyses were performed on the statically stabilized models with pseudo-static coefficients ranging from 0.06 to 0.22. The results revealed that ground anchored slopes generally showed acceptable performances under dynamic loading, while higher axial forces were induced to ground anchors in higher and steeper models. Furthermore, comparing the results of dynamic and pseudo-static analyses showed a good agreement between the two methods’ predictions in the mobilized axial force along the ground anchors. Pseudo-static coefficients were then proposed to replicate dynamic results, considering the slope geometry and dynamic load peak acceleration. The results revealed that higher and steeper stabilized slopes required higher values of pseudo-static coefficients to match the dynamic predictions successfully. The results indicate that pseudo-static coefficient tend to increase with the increase in dynamic load peak acceleration in any given model.

Keywords: Geotechnical Earthquake Engineering; Slope Stability; Seismic Stability; Pre-Stressed Anchors; Pseudo-Static Coefficient.

1. Introduction

Pre-tensioned ground anchors are among the most effective technologies in stabilizing deep excavations due to their efficiency in limiting the excavation wall crown displacement more than other tieback retaining systems such as soil nailing and rock bolts. Therefore, their application has increased worldwide for both temporary and permanent excavation wall and slope stabilizations in recent decades. While this system is mostly designed for temporary stabilization of the cuts before the construction of the permanent retaining structure, it has shown some exemplary performance in earthquake-prone regions such as Athens subway stations, where the anchored wall resisted high magnitudes of ground motions, despite being designed for the minimal acceleration levels [1-3]. These incidents brought more attention to the investigation of dynamic performance pre-tensioned retaining walls.

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One of the first attempts in evaluating the dynamic behavior of pre-tensioned anchored walls was a finite element time history analyses conducted on a three-row anchored wall by Fragaszy et al. (1987) [4]. Their analyses showed that the flexibility of the pre-tensioned anchored system allows it to move in-phase with the surrounding soil, while the out-of-phase displacements of the retaining wall proved to induce high magnitudes of horizontal pressure and horizontal moments to the wall facing [4]. Further numerical parametric analyses conducted by Siller and Frawley (1992) [5] revealed that the normal stiffness of the anchor element plays an essential role in the magnitude of mobilized dynamic load, as it also decreases the dynamic displacements. Siller et al. (1991) also found that the pre-tensioning load is the most effective parameter in limiting the static displacements, while having a very insignificant impact on dynamic displacements [6]. Therefore, dynamic design instructions recommended the anchor element normal stiffness to be adopted beyond the design values resulted in the temporary static design of anchored walls.

Siller and Dolly (1992) [7] conducted a series of numerical seismic investigations on anchored wall higher number of low-capacity anchors. The result of their model revealed that pre-tensioned ground anchor stabilized walls with a higher number of low-capacity anchor elements perform more reliably under seismic loading than those with a fewer number of high-capacity anchors. Furthermore, anchored walls were also found to have more efficient seismic performance over retaining walls, especially in a higher ground acceleration up to 0.5g. Sheet piles stabilized with pretensioned anchors also proved to have acceptable performances under seismic loading with a peak acceleration equal up to half of the earth's gravity while experiencing high magnitudes of horizontal excess pore water pressure developed behind the wall [2].

Due to the complexity of dynamic and seismic analyses, engineers and designers have adopted the pseudo-static approach to predict the structure’s response to dynamic loadings since the mid-1920s. In pseudo-static slope stability analyses, the earthquake effects are represented by constant horizontal force, which is the resultant of the active wedge mass multiplied to pseudo-static coefficient (k_h), applied to the center of the soil active wedge mass, and the static analyses takes place considering the pseudo-static force [8]. Farhangi and Karakopuzian (2020) found that the impact of seismic hazards in natural soil slopes could be reduced by jet grouting by micropiles. They also revealed that a high correlation exists between the soil stiffness represented by (N_1)_{90} in SPT test and the safety against liquefaction caused by strong dynamic loads in coarse-grained materials [9].

Oliaie and Tohodifar (2018) proposed an optimum distance of three times the pile diameter for the best seismic stabilization of slopes with sleeved and unsleeved piles [10]. More recently, Peng et al. (2020) [11] also pointed out the impact of ground motion intensity on the seismic response of rock slopes stabilized with anchors. Their findings revealed that the maximum axial force mobilized across the anchor is near the rock interface, which shows that the structural plane of the slope takes over the seismic response. The seismic performance of retaining walls with compressible inclusions was investigated by Dram et al. (2020) [12]. They applied 15 cycles of sinusoidal dynamic loads with PGA ranging from 0.1g to 0.3g. The findings from this study revealed that compressible tire shreds could decrease the permanent displacement of the retaining walls down to half. A recently-published FEM numerical study by Farrokhzad et al. (2021) on the seismic behavior of nailed excavation walls revealed the importance of nail elements’ spacing and length excavation wall seismic deformation. Moreover, their analyses revealed steeper nail elements could reduce the permanent seismic wall deformation more effectively [13].

Tiwari et al. (2014) reported the outcomes of pseudo-static analyses are critically dependent on the value of k_h, which is usually recommended to be adopted roughly between 0.1 to 0.2 [14]. Komak Panah and Majidian (2013) adopted the finite difference method to predict soil nail walls' seismic behavior. The result of their study showed that by adopting the proper k_h, a good agreement could be observed between numerical seismic and numerical pseudo-static analysis results [15].

Previous studies revealed the flexible performance of anchored walls under seismic loads and the pseudo-static approach’s effectiveness in dynamic slope stability analyses [1-3, 15]. While numerical analyses are powerful tools in computational modeling of geo-mechanical problems, they require considerable computational power and are time-consuming procedures. In the case of dynamic analyses performed in this study depending on the model's dimensions, it took 8 to 32 hours for each analysis. On the other hand, numerical pseudo-static analyses can be performed in several minutes. If an accurate pseudo-static coefficient is implemented is capable of replicating dynamic analysis results. Also, previous studies on pseudo-static analyses suggested using the value 0.1 to 0.2, which is a wide range and can lead to a big difference in the mobilized internal and external forces and dynamic displacements. Additionally, this wide range of pseudo-static coefficients (k_h) neither considers the type of the slope and the supporting retaining structure nor the height and the inclination of the slopes and the seismic load intensity. To this end, this study aims to suggest k_h values for the pre-stressed-ground-anchored retaining structures to enable more accurate prediction of dynamic mobilized anchor forces considering model geometry, including height and slope inclination, as well as ground motion intensity. In order to accomplish this, six numerical anchored excavation models with 10m and 20m height and slope inclinations of 60°, 75°, and 90° are developed and statically stabilized. Then all of these models are subjected to two dynamic loads, each of which having Peak Ground Acceleration (PGA) ranging from 0.2g to 0.6g. Pseudo-static analyses are conducted.
by applying $k_h$ ranging from 0.06 to 0.22. At the end, the results of dynamic mobilized forces along the anchors are bracketed by outcomes numerical pseudo-static analyses to propose the most valid $k_h$ for that specific geometry and PGA. Furthermore, the numerical model is validated against full-scale experimental data in predicting the response of pre-tensioned anchored wall excavation with good agreements. Figure 1 shows a flowchart of the research methodology employed in this study.

![Flowchart of research methodology](image)

**Figure 1. Research methodology flowchart**

### 2. Model Description

All numerical models were developed with the Finite Difference Method (FDM) using FLAC$^{2D}$. An overall of six excavations representing 10m and 20m height excavations with different face angles of 60°, 75°, and 90° with the horizon was developed representing a wide range of excavation geometry with mild to steep face inclinations. Models will be denoted with two-part names as H10-75 representing the model with 10m height and 75° herein.

![Schematic models' geometry and anchors arrangements](image)

**Figure 2. Schematic models’ geometry and anchors arrangements (not scaled)**
After a series of trial and error in developing models with different anchor lengths, horizontal and vertical distances and lock of loads the final arrangements shown in Figure 2 were found to best satisfy the minimum static factor of safety equal to 1.5 recommended by Federal Highway Administration Ground Anchors and Anchored System for permanent application of ground anchors [16]. To simulate a three-Dimensional problem in two dimensions, Donovan et al. (1984) [17] suggested linear scaling of material properties. This simple and convenient approach of distributing elements’ discrete effect over the distance between elements in a regularly spaced pattern. This is accomplished by dividing the element properties by the distance between elements. Therefore, the anchors’ pre-tensioning load was set to 450 kN for all anchors in the design was scaled to 150 kN for 10 meter-height excavations due to 3 meters horizontal spacing of anchors and 180 kN for 20 m excavations due to 2.5 m meters horizontal spacing.

Kulhemeyer and Lysmer (1973) [18] suggested that zone dimensions in a finite difference analysis be one-tenth to one-eighth of the input wavelength’s highest frequency component for proper wave propagation in dynamic analyses. In the present study, the wavelength for H-1 and H-2 harmonic loads were 55 and 46.87 meters, respectively; hence the dimensions of the zones are 25 cm, and 50 cm in the models of 10m and 20m are qualified for the proper wave propagation condition for both models.

2.1. Soil

Tehran alluvium, classified as SM according to the Unified Classification System, was chosen for the analyses. The hyperbolic Duncan-Chang model was chosen to simulate the soil behavior. This nonlinear model considers the stress level effects on the stiffness and strength of the soil [19]. In addition, it provides the possibility of modeling unloading and reloading of the along a different path from the loading path. The tangent Young’s modulus $E_t$, the initial slope of the stress-strain curve in the hyperbolic model, is defined as

$$E_t = \left[1 - \frac{R_f(1 - \sin\varphi)(\sigma_1 - \sigma_3)}{2c \cos\varphi + \sigma_3 \sin\varphi}\right]^2 K \cdot P_a \left(\frac{\sigma_3}{P_a}\right)^n$$

(1)

Where $R_f$ is the failure ratio, $K$ is the initial tangent Young’s modulus factor; $P_a$ is the atmospheric pressure, $\varphi$ is the friction angle of the soil, $c$ indicates the cohesion of the soil, $\sigma_1$ and $\sigma_3$ are major and minor main stresses, and $n$ is the stress influence component [19-20]. Accordingly, the tangent Bulk modulus and Young’s modulus for Unloading-Reloading ($ur$) are expressed as

$$B_t = K_b \cdot P_a \left(\frac{\sigma_3}{P_a}\right)^n$$

(2)

$$E_{ur} = K_{ur} \cdot P_a \left(\frac{\sigma_3}{P_a}\right)^n$$

(3)

Where $K_b$ is the bulk modulus factor, and $K_{ur}$ is unloading-reloading modulus factor. The soil model parameters for Duncan and Chang constitutive model were calibrated and represented [21, 22]. In Table 1 the parameters were derived based on calibration of the model prediction against triaxial and direct shear test results.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$K$</th>
<th>$n$</th>
<th>$R_f$</th>
<th>$\varphi$</th>
<th>$C$ (kPa)</th>
<th>$K_{ur}$</th>
<th>$K_b$</th>
<th>$\gamma$ (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>285</td>
<td>1</td>
<td>0.9</td>
<td>38°</td>
<td>10</td>
<td>600</td>
<td>474</td>
<td>19.7</td>
</tr>
</tbody>
</table>

2.2. Structural Elements

Beam elements, which are capable of resisting axial forces and bending moments, were implemented in order to model reinforced concrete for the excavation facing flexural strength for a 30 cm reinforced concrete wall facing ($EI$) was suggested to be 29.9 MPa [13,15, 23]. Cable elements were used to simulate high resistance steel strands with ultimate tensile strength ($f_u$) of 1860 MPa used by Gazetas et al. 2016 [2].

2.3. Boundary Conditions

Boundary conditions play essential roles in both static and dynamic analyses. Their role is considerably more critical in dynamic studies as the reflection of dynamic waves from boundaries could result in numerical errors in model response. The boundary conditions of the model were taken as full-fixities at the base of the model with vertical rollers on the lateral sides of the model except for models for static and pseudo-static analyses. Free-field boundary conditions in finite difference numerical modeling using FLAC are a practical approach to prevent wave reflection and eliminate the boundary effects in dynamic analyses, which were specified along the model’s lateral edges [24].
2.4. Damping

Using elasto-plastic soil models provides sufficient hysteretic damping when dynamic shear stress exceeds soil plastic yield resistance, and plastic deformations take place. However, further damping is required in an elastic state. Rayleigh damping with a 3% ratio was implemented to the soil profile with the central frequency of damping set to the fundamental frequency of the structure suggested by FLAC manual [24].

3. Loading Description

3.1. Construction Stage (Static) Analyses

Since plastic deformations and stress redistribution affects the total excavation results, the actual excavation sequences should be simulated prior to dynamic and pseudo-static analyses. To this end, after the mesh generation, excavation to the first anchor level is modeled as shown in Figure 3b. It should be noted that the mesh dimensions are finer at regions where the anchors are meant to be installed to provide more accurate calculations. The anchor element installation is shown in Figure 3c, and after that, the pre-tensioning load is applied to the anchor element. As illustrated in Figure 3d, the pre-tensioning load decreases gradually in the bonded portion of the anchor and does to zero at the end of this area while stays constant in the un-bonded area where the soil and anchor element are arranged to have zero interaction. These steps are repeated to the point the desired excavation depth is reached.

3.2. Dynamic Loading

Dynamic loads were chosen in a way that represent the main characteristics of real earthquakes in which the acceleration increases gradually and then decreases towards the end of the earthquake. Horizontal acceleration time histories as presented in Equation 5 were applied to the model's base boundaries.

\[ a = \sqrt{\beta e^{-\alpha t}} \xi \sin 2\pi ft \]  

Where \( f \) is the dynamic load frequency, and \( \alpha, \beta, \) and \( \zeta \) are the coefficients determining the shape and number of cycles in dynamic loading. Based on the parameter values presented in Table 2, harmonic acceleration time histories H-1 and H-2 representing moderate and severe earthquakes, respectively scaled to 1 m/s² acceleration are shown in Figures 4-a and b. In this study, models are subjected to H-1, and H-2 scaled to 0.1g to 0.6g, where g is the acceleration of the gravity (9.81 m/s²).

<table>
<thead>
<tr>
<th>Harmonic load type</th>
<th>( f )</th>
<th>( \alpha )</th>
<th>( B )</th>
<th>( \zeta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-1</td>
<td>5</td>
<td>5</td>
<td>5.75</td>
<td>11.8</td>
</tr>
<tr>
<td>H-2</td>
<td>6</td>
<td>3.1</td>
<td>0.2</td>
<td>11.8</td>
</tr>
</tbody>
</table>
3.3. Pseudo-static Analyses

The pseudo-static approach is a conventional method to simulate the seismic effects of earthquakes on excavations. In the limit equilibrium method, the seismic effects are simulated by a horizontal force \( F_h \) equal to the product of the active soil weight \( W \) and the earthquake horizontal coefficient \( k_h \) as the following:

\[
F_h = k_h \cdot W
\]  

(6)

If \( W \) is broken into its constituents, mass (m) and g, then Eq. 6 could be rewritten as

\[
F_h = (k_h \cdot g) \cdot m
\]  

(7)

The pseudo-static analyses were conducted by implementing this concept by changing the magnitude and angle of the applied gravity acceleration \( g \) in FLAC and converting it to virtual gravity acceleration \( g' \). Figure 5 further illustrates how \( g' \) results from based on \( g \) and \( k_h \) [15].

![Figure 5. Acceleration resultants of the virtual gravity](image)

3.4. Model Validation

The anchored excavation numerical model developed with FLAC\textsuperscript{2D} was validated against the full-scale instrumented anchored wall constructed and tested at the National Geotechnical Experimentation Site on the riverside campus of Texas A&M University. This wall was 60 m in length and 7.5 m in height. It was built by driving H piles in a line on 2.44-m center for one part of the wall and by drilling and grouting, H piles in a line on 2.44-m center for the other part of the wall. As it is shown in Figure 6a, half of the anchored wall was stabilized with only one row of anchors, while the other half had two rows of anchor reinforcement. The steel H piles were HP 6 * 24 section, 9.15 m in length, embedded 1.65 m below the bottom of the excavation. The wall facing between the H-piles was stabilized by installing wood lagging boards.

The soil profile in the site was a 13-m-thick layer of medium dense, fine silty sand deposited in a river environment; 50,000 years ago, and underlain by a 40-million-years-old hard shale. The engineering properties and the geology of this sand deposit have been determined in detail as part of the National Geotechnical Experimentation Site program [25, 26]. The following average properties of the sand are a total unit weight of 18.5 kN/m\textsuperscript{3}, standard penetration test blow count increasing from 10 blows per 0.3 m at the surface to 27 blows per 0.3 m at the bottom of the piles, borehole shear friction angle of 32\textdegree{} with no cohesion, cone penetration test point resistance of 7 MPa, PMT modulus of 8 MPa, and PMT limit pressure of 0.5 MPa. The water level is 9.5 m below the top of the wall.

The two-row anchor wall was used to calibrate the numerical model. The same numerical values and parameters used by Briaud and Lim (1999) in FEM modeling of the wall was implemented in the FDM model as shown in Table. 3.
Table 3. Parameters used in numerical model of Texas A&M University anchored wall [25]

<table>
<thead>
<tr>
<th>Data</th>
<th>Parameters</th>
<th>Values</th>
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<tbody>
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<td>Soil</td>
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</tr>
<tr>
<td></td>
<td>$n$</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>$R_f$</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td>$\varphi$</td>
<td>32°</td>
</tr>
<tr>
<td></td>
<td>$C$(kPa)</td>
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<tr>
<td></td>
<td>$K_{so}$</td>
<td>1200</td>
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<tr>
<td></td>
<td>$K_0$</td>
<td>272</td>
</tr>
<tr>
<td></td>
<td>$\gamma$ (kN/m$^3$)</td>
<td>18.5</td>
</tr>
<tr>
<td></td>
<td>$K_{u}$</td>
<td>0.65</td>
</tr>
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<td>Anchors</td>
<td>Unbonded length (m)</td>
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</tr>
<tr>
<td></td>
<td>Bonded length (m)</td>
<td>7.3</td>
</tr>
<tr>
<td></td>
<td>Lock-off load row 1 (kN)</td>
<td>183.2</td>
</tr>
<tr>
<td></td>
<td>Lock-off load row 1 (kN)</td>
<td>160</td>
</tr>
<tr>
<td></td>
<td>Stiffness (kN.m)</td>
<td>19.846</td>
</tr>
<tr>
<td></td>
<td>Angle of Inclination ($\beta$)</td>
<td>30°</td>
</tr>
<tr>
<td>Wall</td>
<td>Wall height (m)</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td>Length of soldier-pile(m)</td>
<td>9.15</td>
</tr>
<tr>
<td></td>
<td>Pile Embedment length (m)</td>
<td>1.65</td>
</tr>
<tr>
<td></td>
<td>Flexural stiffness, $EI$ (kN.m$^2$)</td>
<td>11.620</td>
</tr>
<tr>
<td></td>
<td>Axial stiffness $EA$ (kN)</td>
<td>$1.47 \times 10^6$</td>
</tr>
<tr>
<td></td>
<td>Elasticity modulus (GPa)</td>
<td>210</td>
</tr>
</tbody>
</table>

The wall displacement, moment, and axial force resulted from the numerical model, and the field test is depicted in Figure 7. As Figure 7a demonstrates, both numerical and experimental approaches show the maximum displacement at the wall crown and the minimum at the wall base with an acceptable agreement. The numerical model also shows good
agreement in predicting the axial force and moment along the wall. Both models confirm that the maximum moment is developed at the anchor level, while the maximum axial force is generated at the lower anchor level.

![Figure 7](image.png)

**Figure 7** Comparison of numerical results and field data

4. Results

4.1. Dynamic Analyses Results

The axial forces mobilized in anchors at the end of dynamic analyses, and pseudo-static analysis served as the basis for comparing the dynamic and pseudo-static analyses. As shown in the Figure 8 the axial load in the anchors also demonstrates the harmonic behavior of the dynamic load applied to the model during the excitation time. The stable mobilized load at the end of the dynamic excitation remains in the anchor element as the dynamic load. The axial load time history of the anchor element demonstrated the same frequency as the dynamic load applied to the model.

![Figure 8](image.png)

**Figure 8.** Anchor element axial force time history under load H-2

Figures 9 through 11 represent the anchors’ dynamic forces for 10m models subjected to H-1 and H-2 dynamic load with increasing PGA from 0.2g to 0.6g. As expected, the dynamic loading resulted in an increase in mobilized force in ground anchors. The increase of the PGA results in a higher magnitude of mobilized dynamic forces in ground anchors. The slopes’ angle also proved to play an important role in the value of mobilized force, and the anchors in vertical models (i-90) experienced the highest dynamic force increase, while lower magnitudes of dynamic force were generated in i-75s and i-60s, respectively. Dram et al. (2020) [12] observed the spike pattern in the horizontal earth pressure at the one-sixth of the bottom of wall height in their FEM model which also emerges here as the dynamically mobilized anchor.
force. This pattern is more noticeable in the results of 20m models in Figures 12 through 14. The lowest anchor, located at the one-sixth of the bottom of wall height, undergoes a much higher dynamic load than the anchor elements above that. Farrokhzad et al. (2021) [13] also reported the same pattern in soil nails, but the percentage of the mobilized dynamic force at the lowest soil nail is considerably less than what was observed in this study. The reason for that could be the higher capacity of ground anchors compared to soil nail elements as well as higher PGAs implemented in this study compared to Tabas earthquake time history applied to the FEM soil nail model.

In terms of the impact of the PGA on the horizontal earth pressure and mobilized anchor force, it is observed that the axial troops in anchors in this study are more impacted by the increase in the PGA comparing to the horizontal earth pressure reported by Dram et al. (2020) [12]. It can be concluded that the flexural stiffness of the retaining structure impacts the dynamic horizontal pressure and forces developed at the retaining wall. However, it should be mentioned that the retaining structure height may also be an important factor in this regard since it is noticed that in the 20m models, the impact of PGA increases and the anchors tend to undergo higher tension with ground motion intensity increased compared to what observed in 10m models in Figures 9 through 11.

Also, it was observed that with the suggested minimum factor of safety of 1.5 no failure took place in any of the models which validates the recommended Factor of safety by FHWA. However, should any failure or rupture have happed in the dynamic or pseudo-static analyses suggesting any valid coefficient became impossible. It also validates authors choice for material strength and parameters to develop dynamically-resistant enough models to resist dynamic loads to this intensity.

Figure 9. Dynamic axial forces of the anchors along the height of the model H10-i60

Figure 10. Dynamic axial forces of the anchors along the height of the model H10-i75

Figure 11. Dynamic axial forces of the anchors along the height of the model H10-i90
Similar to 10 models, dynamic forces along the anchors increase with PGA and have higher values for more steep excavations. The anchors’ dynamic loads in each model increase from the top to the bottom of the excavation, with more intensity the lower 3 anchors with the maximum mobilized dynamic force in the lowest anchor for all models. The highest difference of dynamic forces generated with different Ground acceleration was generated from PGA=0.4g to PGA= 0.5g in both 10 and 20 m models.

**Figure 12.** Dynamic axial forces of the anchors along the height of the model H20-i60

**Figure 13.** Dynamic axial forces of the anchors along the height of the model H20-i75

**Figure 14.** Dynamic axial forces of the anchors along the height of the model H20-i90

In this study, anchors were designed and modeled to resist the highest mobilized dynamic load. Therefore, the cross-section of all the ground anchors set to withstand 482 kN mobilized in the bottom anchor of model H20-i90, under dynamic load H=2 with 0.6g PGA.

### 4.2. Pseudo-static Analyses Results

Pseudo-static analyses were conducted to replicate the dynamic force mobilization in anchors. The dynamic forces were the criterion for selecting the values of $k_h$ for models so that the axial forces induced in pseudo-static analyses would totally encompass the axial forces induced under all dynamic loading scenarios. Therefore, 10m models were subjected to pseudo-static coefficients $k_h =0.06$ to $k_h =0.2$, and 20-m-models were subjected to $k_h =0.12$ to $k_h =0.22$ under pseudo-static analyses.

Axial forces induced in pseudo-static analyses showed a similar pattern to dynamic axial forces along the models’ depth, as shown in Figures 15 through 20. Similar to dynamic analyses, pseudo-static induced forces in anchors increase
with depth, with the maximum values in the lowest anchors in all models. Three anchors at the bottom in 20m models witnessed drastic changes compared to anchors above them in each model, and an increase in pseudo-static coefficient ($k_h$) led to raising in induced pseudo-static axial force in anchors in models. The same behavior in the mobilization of noticeably higher axial force in lower anchors is also seen in pseudo-static analysis results. The top four anchors in 20-m-height slopes in both dynamic and pseudo-static analyses revealed the least dependency on either PGA or pseudo-static coefficient and PGA, while in 10-m-height models, the anchor forces increase more noticeably top and middle anchors, which is due to the higher horizontal displacement of the anchors in 10-m-model comparing to the 20-m-model and highlights the role of horizontal spacing in the dynamic and pseudo-static response of anchored walls since the vertical spacing were chosen the same for both models in the design and modeling. $k_h=0.22$ causes the highest deviation from the lower pseudo-static coefficients in 20m models. These same pattern behaviors are observed in dynamic analyses under PGA higher than 0.4g. This implies that since earthquakes with PGA higher than 0.4g are not considered probable scenarios for earthquakes $k_h$ higher than 0.2 is far from recommended values for retaining structures pseudo-static analyses.

Figure 15. Pseudo-static axial forces of the anchors along the height of the model H10-i60

Figure 16. Pseudo-static axial forces of the anchors along the height of the model H10-i75

Figure 17. Pseudo-static axial forces of the anchors along the height of the model H10-i90

Figure 18. Pseudo-static axial forces of the anchors along the height of the model H20-i60

Figure 19. Pseudo-static axial forces of the anchors along the height of the model H20-i75

Figure 20. Pseudo-static axial forces of the anchors along the height of the model H20-i90
4.3. Pseudo-static Coefficient

Based on the induced axial forces in pseudo-static and dynamic analyses, \( k_h \) values to replicate the dynamic induced axial force with pseudo-static axial forces along the depth of each model are presented in Table 4. Minimum and maximum values of \( k_h \) that reproduced the dynamic response of the model were then averaged to have the unique pseudo-static coefficient in each case.

\[ k_h = \beta \frac{a_{\text{max}}}{g} \]  

(8)

\[ \beta = \begin{cases} 
0.25 & \text{for } H=10 \text{ m, } i=60^\circ \\
0.38 & \text{for } H=10 \text{ m, } i=75^\circ-90^\circ \\
0.43 & \text{for } H=20 \text{ m, } i=60^\circ-90^\circ 
\end{cases} \]  

(9)

Table 4. Variation of the pseudo-static coefficients in the models

<table>
<thead>
<tr>
<th>Excavation height (m)</th>
<th>Excavation inclination (°)</th>
<th>PGA(g)</th>
<th>Min. ( k_h )</th>
<th>Max ( k_h )</th>
<th>Ave. ( k_h )</th>
<th>( \beta )</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>60</td>
<td>0.2</td>
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<tr>
<td></td>
<td>0.4</td>
<td>0.08</td>
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<td></td>
<td>90</td>
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<td>0.16</td>
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<td>0.20</td>
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<td>0.22</td>
<td>0.20</td>
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</tr>
</tbody>
</table>
5. Conclusion

The responses of anchored excavations under dynamic loading using harmonic time history were compared against pseudo-static analyses to examine the latter approach’s ability to predict the mobilized dynamic forces in ground anchors reliably. The finite difference numerical model developed by FLAC$^{2D}$ demonstrated good agreements with full-scale experimental results and proved to be effectively capable of simulating the stabilized excavation with pre-tensioned anchors. The applied dynamic loadings induced excess axial forces in pre-tensioned lock-off force that increased throughout the excavation depth with considerable higher magnitudes at the lower one-third of the wall height. Both dynamic and pseudo-static approaches induced axial forces in pre-tensioned lock-off load in the same manner, and the pseudo-static approach implemented in this study successfully replicated the dynamic axial forces induced in pre-tensioned anchors. Anchor spacing was found to be an important factor in mobilization of both dynamic and pseudo-static forces. Comparing the outcomes of this study with the retaining walls also revealed that higher wall flexural stiffness decreases the mobilized horizontal forces and pressure behind the retaining structure. Suggested pseudo-static coefficients effectively predict the values of the induced axial forces in anchors based on the maximum acceleration of applied dynamic load as well as the excavation geometry. An increase in the $\dot{a}_{\text{max}}$ leads to a decrease in the value of the coefficient $\beta$ represented for each model. This reveals that although the pseudo-static coefficients were found to increase with the acceleration level of dynamic loads, this increase would be slighter comparing to the maximum acceleration level. Considering the severe effects of earthquakes on excavation toes, it is recommended that more reinforcement, including increasing un-bonded length of anchors or increased pre-tensioning load, should be considered for these areas.

6. Declarations

6.1. Author Contributions

Conceptualization and methodology, A.S.R.O., and M.O.; software, A.S.R.O.; validation, A.S.R.O.; formal analysis, A.S.R.O.; investigation, A.S.R.O.; writing—original draft preparation, A.S.R.O.; writing—review and editing, A.S.R.O., M.O. and H.H.; visualization, A.S.R.O. and H.H.; supervision, M.O. and H.H.; All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

The data presented in this study are available in article.

6.3. Funding

The author(s) received no financial support for the research, authorship, and/or publication of this article.

6.4. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Effect of Polypropylene Fibers on moisture Susceptibility of Warm Mix Asphalt

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Abstract

Warm Mix Asphalt (WMA) is a modern energy-saving process that uses environmentally friendly materials, has lower mixing and compaction temperatures, and uses less energy and releases less contaminants than conventional hot mix asphalt. Moisture damage poses one of the main challenges of the material design in asphalt pavements. During its design life, the asphalt pavement is exposed to the effect of moisture from the surrounding environment. This research intends to investigate the role of the polypropylene fibres for modifying the moisture susceptibility for the WMA by using different percentages of polypropylene (namely 2, 4, and 6%) by weight of the binder of the control mixture (WMA). In this paper, the physical characteristics of the asphalt cement, Marshall properties, Tensile Strength Ratio (TSR) and Index of Retained Strength (IRS) were determined to establish the effect of the polypropylene on the moisture susceptibility of the WMA. The results displayed that the modification of the AC with polypropylene caused an increase in the optimum asphalt content by 1.03, 3.09, and 11.3%, with the addition of 2, 4 and 6% of the P.P., respectively. The moisture resistance of the asphalt mixture was enhanced by adding the P.P., according to the rise in the Tensile Strength Ratio (TSR) and Index of Retained Strength (IRS) values. The TSR value showed 9.4, 18.2 and 19.5% increase when the P.P. increased from 0.00 to 0.02, 0.04, and 0.06, respectively; besides, the IRS showed improvement with the addition of the P.P. to the WMA.

Keywords: Warm Mix Asphalt (WMA); Indirect Tensile Strength (ITS); Tensile Strength Ratio (TSR).

1. Introduction

Warm mix asphalt (WMA) used in road construction is a new energy saving method and provides an environmentally friendly protective step via reduced mixing and compaction temperatures and relatively lower energy consumption compared with the use of the conventional Hot Mix Asphalt (HMA) [1]. The production of WMA differ than that of (HMA) since the mixing temperature as well as compaction temperature approximately lower than that of HMA by 15-40 degree Celsius depending on the type of additives adopted to produce WMA [2]. Over the recent years, environmental protection is gaining recognition as an important factor in transport engineering, particularly in asphalt production. Even with the popular use of HMA in different countries, some researchers recommend switching to a different technology that involves lower temperature production for the manufacture of asphalt mixtures via the use of WMA; this technology is currently being employed in the United States and European Union countries [3].

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A key durability issue associated with the ability of asphalt pavement to resist the effects of water, without significant deterioration in the pavement. The moisture damage is commonly referred to as loss of the adhesion bond at the asphalt-aggregate interface or loss of the cohesion of asphalt binder [4]. Susceptibility of moisture in asphaltic pavement might be considered as a serious defect that caused stripping, and development of other problems including fatigue cracking and permanent deformation [5]. Stripping issues are the main problems related to the WMA, as the lower production temperature will cause incomplete drying of the water entrapped in the aggregate, which subsequently will weaken the interface and adhesive characteristics between the aggregate and asphalt binder, resulting in the separation of the aggregate from the asphalt after the application of traffic loads on the asphalt mixture [6]. In spite of some moisture damage issues being reported, particularly those associated with foaming and some chemical technologies in case of WMA. In last years, different additives types have been used by researchers to improve the moisture resistance, one of which is polypropylene (P.P.) [7].

Polypropylene fibres are used extensively as a reinforcing agent in the AC. The P.P. fibres provide (3) dimensional reinforcement to concrete, ensuring that the concrete becomes tougher and more durable. These fibres can’t substitute the mesh of wire reinforcement. P.P. fibres function of the secondary reinforcement and thus makes it more economical by partially replacing the steel fibers. P.P. fibres are a vital component of high-performance concrete. In the United States, P.P. Fibres were also employed as a modifier in the AC [8]. These P.P. fibres are hydrophobic, meaning they do not absorb water. Therefore, when mixed with the concrete they need to be mixed only long enough to ensure dispersion within the concrete mixture. The fibre length normally recommended is linked to the nominal maximum size of the mixture aggregate [9]. Polypropylene is one of the polymers that finds the most extensive use across the world because of its widespread availability and low manufacturing cost. Polymers can improve the properties of the mixture because it offers the possibility of producing mixtures that can resist rutting, cracking and moisture damage [10].

2. Materials and Methods

The first stage in the experimental plan was to determine the optimum asphalt content of the control mix (WMA) without the addition of the polypropylene, and the second stage was to determine the optimum asphalt content of the WMA with polypropylene for each percentage of polypropylene, depended on the Marshall characteristics. The OAC specified from these tests were then used to produce mixtures for the indirect tensile tests to evaluate the TSR and compression strength tests to evaluate the IRS. The test program of this work is presented by a flow chart as shown in Figure 1.

2.1. Asphalt Cement

One type of AC of (40-50) obtained from the Al-Durrarah Refinery can be used in this research. The tests conducted on the AC confirmed that its properties complied with the specifications of the SCRB [11]. Table 1 shows the physical properties of this type of AC.

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Designation No.</th>
<th>Value</th>
<th>SCRB Specification</th>
</tr>
</thead>
<tbody>
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<td>Penetration, 1/10 mm, 25°C, 100 g, 5 sec</td>
<td>D5</td>
<td>44</td>
<td>40-50</td>
</tr>
<tr>
<td>Softening Point, ° C (ring &amp; ball)</td>
<td>D36</td>
<td>50</td>
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</tr>
<tr>
<td>Ductility, cm (25°C, 5 cm/min)</td>
<td>D113</td>
<td>125</td>
<td>&gt;100</td>
</tr>
<tr>
<td>Specific Gravity, 25°C</td>
<td>D70</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>Flash Point, ° C, Cleveland open cup</td>
<td>D92</td>
<td>269</td>
<td>&gt;232</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Designation No.</th>
<th>Value</th>
<th>SCRB Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retained Penetration of Residue, % (25°C, 100 g, 5 sec)</td>
<td>D5</td>
<td>60%</td>
<td>&gt;55</td>
</tr>
<tr>
<td>Ductility of Residue, cm (25°C, 5 cm/min)</td>
<td>D113</td>
<td>83</td>
<td>&gt;25</td>
</tr>
</tbody>
</table>

2.2. Aggregate

In this work the aggregate was obtained from the Asphalt Plant of Hammurabi in the city of Ramadi, which was basically a large rock broken using the company crushers to obtain the required gradation. The sizes of the coarse aggregate were in the range from ¾ in to No. 4. The fine aggregate size was in the range from No. 4 to No. 200, as clearly defined, in accordance with the requirement of the SCRB (2003) [12]. The physical properties of the aggregate are listed in Table 2.

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Designation No.</th>
<th>Coarse Aggregate</th>
<th>Fine Aggregate</th>
<th>SCRB R/9 2003</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk specific gravity</td>
<td>ASTM C127 and C128</td>
<td>2.647</td>
<td>2.635</td>
<td>----</td>
</tr>
<tr>
<td>Percent water absorption</td>
<td>ASTM C127 and C128</td>
<td>0.13</td>
<td>0.524</td>
<td>----</td>
</tr>
<tr>
<td>Percent wear (Los-Angeles Abrasion)</td>
<td>ASTM C131</td>
<td>19.7</td>
<td>----</td>
<td>30 Max</td>
</tr>
</tbody>
</table>
2.3. Mineral Filler

The filler is a material that can pass through sieve No. (0.075 mm), and is usually used to enhance the properties of mixture by reducing the plasticity, increasing the viscosity and decreasing the change of volume. In this work, limestone was the filler used. It was sourced from the lime factory in the governorate of Karbala (Iraq). Table 3 represent the physical properties of the mineral filler that used in this work.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Result</th>
<th>SCRIB Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.73</td>
<td>-</td>
</tr>
<tr>
<td>Passing Sieve No. 200 (0.075 mm), %</td>
<td>94</td>
<td>70-100</td>
</tr>
</tbody>
</table>
2.4. Zeolite

Aspha-min powder was used as the additive for the WMA production in the laboratory. Aspha-min is industrialized synthetic sodium aluminium silicate, or enhanced, known popularly as zeolite. This zeolite is hydrothermally crystallized. Eurovia’s Aspha-min® retains about 21% water by mass and is released at temperatures above 100° C. The chemical characteristics for the Aspha-min are presented as shown in Table 4.

Table 4. Chemical composition of zeolite

<table>
<thead>
<tr>
<th>Chemical composition</th>
<th>Result %</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>32.2</td>
</tr>
<tr>
<td>Al2O3</td>
<td>28.5</td>
</tr>
<tr>
<td>Na2O</td>
<td>15.3</td>
</tr>
<tr>
<td>L.O.I.</td>
<td>24.0</td>
</tr>
</tbody>
</table>

2.5. Polypropylene (P.P.)

Polypropylene is a commodity polymer extensively used and known for its good process ability, integral hinge property, low cost, high softening point, low density and good properties of mechanical. Polypropylene fibres are manufactured by the polymerization of P.P. as a linear polymer and termed by its abbreviated form, P.P. The physical and mechanical characteristics of the P.P. according to the manufacturing company are listed in Table 5.

Table 5. Physical Properties of Polypropylene Fibre

<table>
<thead>
<tr>
<th>Type of Properties</th>
<th>Specification of Polypropylene</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw material</td>
<td>Virgin Polypropylene</td>
</tr>
<tr>
<td>Fibre type</td>
<td>Monofilament</td>
</tr>
<tr>
<td>Melting point (°C)</td>
<td>160-170</td>
</tr>
<tr>
<td>Acid and alkali resistance</td>
<td>Strong</td>
</tr>
<tr>
<td>Length (mm)</td>
<td>12</td>
</tr>
<tr>
<td>Break elongation</td>
<td>5%</td>
</tr>
<tr>
<td>Fibre diameter</td>
<td>(20 – 30) µm</td>
</tr>
<tr>
<td>Density (gm/cm³)</td>
<td>0.01</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>450</td>
</tr>
<tr>
<td>Water absorption</td>
<td>No</td>
</tr>
</tbody>
</table>

3. Selection of Aggregates Gradation

According to General Specification for Roads and Bridges (SCRB/R9, 2003), the nominal maximum size of aggregate is 12.5 mm for the wearing course. Selected aggregates gradation for the wearing course can be showed in Figure 2.

![Figure 2. Specification Limits and Selected Gradation for Wearing Course](image-url)
4. Marshall Mix Design

The procedure includes the preparation, compaction and testing of the 101.6 mm diameter and 63.6 mm height cylindrical bituminous specimens of paving mixture using the Apparatus of Marshall, in accordance with ASTM 6926-04 (2010) [13].

4.1. Preparation of WMA

Sieve No. 4 functions as the boundary between the coarse and fine aggregates. This is where the materials retained for this sieve form the coarse aggregate, while the materials passing through it, form the fine aggregate. These materials are washed and at 110°C dried, and separated into sieves after cooling. The mineral filler such as (Limestone dust) was used, that could pass through sieve No. 200. To prepare the WMA, according to the Iraqi standard specifications, the material was recombined with the filler to meet the required gradation. The aggregate mix was heated to 125°C. Also, the asphalt was added to the aggregate directly after adding the zeolite. Figure 3 shows the addition of the zeolite to the aggregate, while Figure 4 indicates the addition of the binder to the prepared warm mix asphalt.

![Figure 3. Shows the process of the addition of the binder to the prepared warm mix asphalt.](image)

![Figure 4. Binder addition to the aggregate and zeolite blend](image)

4.2. Preparation and Testing of ITS Specimens

For evaluating the susceptibility of moisture in asphalt concrete, specimens with dimensions of 101.6 mm diameter and 63.6 mm height are used. Specimens are prepared according to AASHTO, T.283 (2007) [14], left to cool at room temperature for (24 hours) and then extracted from the moulds. The desired No. of blows is determined to obtain test specimens with approximately 7±1% of A.V. using different No. of blows for each face. Figure 5 shows the No. of blows versus percentage of A.V. for the control mixture (WMA).
The indirect tensile strength has been determined by applying the following Equation 1:

\[ ITS = \frac{2000 \text{Pult}}{\pi t D} \]  

(1)

Where; ITS = indirect tensile strength (kPa); Pult = maximum load (N); t = specimen thickness (mm); D = specimen diameter (mm).

The TSR is computed as Equation 2:

\[ TSR = \left(\frac{S_1}{S_2}\right) \times 100 \]  

(2)

Where; TSR = tensile strength ratio (%); S1 = mean indirect tensile strength of the wet subset (kPa); S2 = mean indirect tensile strength of the dry subset (kPa).

5. Index of Retained Strength Test (IRS)

This test method includes the measurement of the loss of compressive strength due to the action of water on the compacted bituminous mixtures containing the asphalt cement. This test method is useful as an indicator of the susceptibility to the moisture of the compacted bitumen-aggregate mixtures and fully covered [9]. Four sets of cylindrical specimens of 4 inches height and 4 inches diameter 101.6×101.6 mm were prepared by compressing the asphalt mixture until the specimen achieved the required height 101.6 mm. The first set was used as the control mixture (WMA) without the added polypropylene fibre, while the other three sets included the added 2, 4 and 6% of the polypropylene fibres by weight of the bitumen in the control mixture, respectively, and each set comprised 6 specimens. The (IRS) was calculated according to AASHTO; and according to SCRB/R9 (2003). The minimum IRS value should be 70%.

\[ IRS \% = \left(\frac{S_2}{S_1}\right) \times 100 \]  

(3)

Where; S1 = compressive strength of dry specimens (kPa); S2 = compressive strength of immersed specimens (kPa).

6. Results and Discussion

6.1. Marshall Stability

Stability is a property for a mixture of asphalt, which indicates its resistance to rutting, while the high stability refers to the increased stiffness of the mixture. High stiffness of the mixture of asphalt indicates good resistance under loadings of traffic. for long-term performance, the lower flexibility of the bitumen mixture is essential, the high stiffness is not recommended because of the potential of the thermal the Marshall stability of the mixtures containing polypropylene was the higher degree of cracking in the future [15], as shown in Figure 6, than that of the control mixtures. The specimens containing polypropylene of (0.02, 0.04, and 0.06 by weight of the binder) had higher values of stability than did the mixtures control by 3.81, 18.1 and 23.8%, respectively. Significant improvement in stability was noted with the increase in the polypropylene content. The scientific interpretation for the increase in stability was the functioning of the polypropylene as "reinforced concrete" when cracks appeared in the asphalt mixture, thus offering resistance to the propagation of the developing cracks.
6.2. Voids in Total Mixture (A.V.)

The A.V. in the total of mixture are an important parameter that can influence the mixture durability of asphalt. According to Tapkin (2007) [8], (3-5%) of the A.V. is sufficient to avoid the bleeding, which occurs due to the low A.V. content less than (3%) in the mixture of asphalt; however, when the high A.V. content is present (exceeding 5%) the mixture of asphalt becomes less durable against the effects of fatigue and moisture. Air voids is the ratio (expressed in %) between the volume of small A.V. between the coated particles and total volume of the mixture [16].

The results showed that the air voids increase in the mixtures that contain polypropylene by 31.6% after the addition of 2% polypropylene; by 31.9% after the addition of 4% polypropylene; and by 0.88% after the addition of 6% polypropylene, compared with the control mixture. The change in the A.V. with the addition of P.P. is shown in Figure 7.

6.3. Tensile Strength Ratio

This test used to evaluate the susceptibility of moisture damage in the AC, only samples having the dimensions of
101.6 mm diameter and 63.6 mm height were selected. The desired No. of blows was determined to obtain test specimens possessing approximately (7%) of air voids, using different numbers of blows per face. The TSR was used to predict the susceptibility of moisture. The recommended limit 80% for the TSR is used to distinguish between the moisture susceptible mixtures and moisture resistant ones. High (TSR) values indicate that the mixture will probably perform better in resisting susceptibility of moisture.

All the results of the TSR comply with the reported findings of earlier researchers, where the rise in (% P.P.) improved the moisture resistance, concurring with Hamedi et al. (2018) and [17]. Also, these results are consistent with the findings of some researchers, such as Mawat (2019) and Ali (2020) [18, 19], as they found that when other types of fibres, such as (carbon fibres and ceramic fibres) are used, the resistance of the asphalt mixtures to moisture damage significantly improved. Furthermore, another researcher showed that the AC modification with nano-clay with montmorillonite led to increase moisture resistance of mixture of asphalt according to the increase in IRS and TSR [20]. Moreover, other local materials were used, such as (Reed Netting Reinforcement) these materials, which increased the resistance of the asphalt mixture to stripping, and thus increased its resistance to moisture damage [21].

The results of the addition of the P.P. reveal that this additive material exerts a positive effect on the amount of the TSR in all of the percentages used in this research. The addition of (2 and 4) % of these materials significantly increases the TSR. While this increase is not significant after 6% of the additive is added (in comparison to the addition of 4% of the additive), it represents that an increase of higher than 4% of this material is not logical because it will only raise the cost of running the mixture; another reason is that no positive effect in terms of increasing resistance against susceptibility of moisture is observed when compared with the samples made with 4%. Consequently, an additional 4% of the P.P. can be considered as the optimal dosage of incorporation. The results showed in this work suggest that the addition 4% of the P.P. material significantly improves the asphalt mixtures resistance against susceptibility of moisture, in comparison with the control samples. The Using additional 4% of the P.P. induced the TSR index, considered the practical index for susceptibility of moisture in executive projects, to exceed the minimum recommended value in most regulations 80%, and thus problem of susceptibility of moisture in the samples could be removed from in terms of regulations.

Although addition 6% of P.P. has also further improved resistance of asphalt mixture against susceptibility of moisture, the difference between the samples with added 4 and 6% P.P. is not significant. Otherwise, the material costs and issues with modification of asphalt are higher in the specimens with the added 6% P.P. Therefore, it appears that the addition of 4% P.P. can be recommended as the optimum concentration of this substance as an anti-stripping additive in the mixtures of asphalt.

6.4. Index of Retained Strength Test (IRS)

IRS was used to evaluate the Compressive Strength (CS) of the mixture of asphalt under the effect of moisture to assess the resistance of the mixture of asphalt to moisture damage. According to SCRB/R9 (2003), the minimum permissible value of IRS is 70 %, which is considered to be the value that indicates whether the mixture is susceptible to moisture or not. The IRS obtained is the percentage of the (CS for conditioned specimens to that of the unconditioned ones). The (ISR) increased by 8.3, 16.7, and 19.4% with the addition 2, 4, and 6% of the P.P., respectively, compared to the control mixture. For the same reasons recorded in the test of the (TSR), the addition of
the P.P. raised the resistance of the asphalt mixture for compression under the effect of moisture, which resulted in the increased I.R.S and enhanced the resistance to moisture, where the resistance to moisture of the mixture of asphalt rises with the increase in the I.R.S. All the results of the I.R.S agree with the reported findings of some researchers, where the rise in P.P. % improved the moisture resistance, agreeing with Mawat (2019) [5].

![Figure 9. Index of Retained Strength for the different mixture types](image)

7. Conclusions

- The properties of the asphalt cement are improved through the use of the WMA at lower temperatures by the adding the zeolite additive.
- Reductions in the compaction temperatures and mixing are important for lowering energy expenditure, as well as the emissions.
- The Marshall stability improved with the addition of the P.P., where it escalated by 3.81, 18.1 and 23.8% after adding the polypropylene (0.02, 0.04, and 0.06%) by weight of the binder.
- The tensile strength ratios improved after the addition of the P.P., which indicated that the added P.P. reduced the moisture susceptibility of the asphaltic mixtures. The TSR values increased by 9.4, 18.2, and 19.5% when the P.P. was increased from 0.00 to 0.02, 0.04, and 0.06, respectively.
- The addition of 2 and 4% of the P.P. significantly raised the TSR. While this increase after adding 6% of the additive when compared with adding 4% of the additive is not significant, it implied that any increase exceeding 4% of this material is not logical, because it will only raise the cost of running the mixture.
- The IRS improved after the P.P. was added, indicating that the use of the P.P. reduced the moisture susceptibility of the asphaltic mixtures.

8. Declarations

8.1. Author Contributions

Conceptualization, S.F.M. and M.Q.I.; writing—original draft preparation, S.F.M. and M.Q.I.; writing—review and editing, S.F.M. and M.Q.I. All authors have read and agreed to the published version of the manuscript.

8.2. Data Availability Statement

The data presented in this study are available in article.

8.3. Funding

The Ministry of Higher Education and Scientific Research, as well as Civil Engineering Department, College of Engineering, Baghdad University, sponsored this study.

8.4. Conflicts of Interest

The authors declare no conflict of interest.
9. References


Preloading Model on Soft Soil with Inclusion Thermal Induction Vertical and Incline Types

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Abstract

Soft clay has a relatively low subgrade bearing capacity. The aim is to obtain physical values, mineralogy, mechanical strength values, values for reduction. The research method used is preloading in a test tube measuring 50×70×150 cm. Each cycle of preloading and thermal induction used a fixed load of 0.015 kg/cm². Thermal induction is given vertically and obliquely with temperature variations of 100, 200, 300, and 400 °C. The main observation point is a distance of 15 cm from the center of the induction. At 400 °C inclined induction, the water content is 17.36% (from the initial water content of 59.07%), the soil cohesion is 21.75. kN/m², the value of unconfined compressive strength is 67.72 kN/m², the highest modulus of elasticity is 4593 kN/m², and the decrease is 5.13 cm. XRD, SEM, EDS results before heating showed mineralogy 0 (65.06%), Ca (13.30%), Na (3.64%), Mg (2.15%), Al (6.63%), Si (8.52%), Sn (0.70%) and did not change significantly after heating at 400 °C. The results after heating included 0 (58.39%), Ca (14.09%), Na (0.72%), Mg (1.16%), Al (6.63%), Si (14.72%), Sn (2.54%). The novelty obtained is to change very soft conditions became medium conditions.

Keywords: Thermal Induction; Preloading; Soft Clay; Modulus Elasticity; Mineralogy.

1. Introduction

The main problem of infrastructure development in soft clay is the relatively low bearing capacity of the base soil and the relatively large compaction of the base soil which lasts relatively longer. To overcome the problem of infrastructure development on soft clay soil, it must be improved or ground improvement to make it stronger or gain strength. One of the soil improvement methods, namely the combination method of preloading and thermal induction, is quite an interesting repair method because the method of implementation is simple and still not widely studied in Indonesia at this time [1].

The soil improvement methods that have been used today are varied such as preloading, prefabricated vertical drain (PVD), electro-osmosis, vacuum consolidation, lightweight fill, stone column, jet grouting, lime columns, fracture grouting, ground freezing, vitrification, electrokinetic treatment, electro heating, and thermal induction. However, the combination method of preloading and Thermal Induction is a method of repair that is quite interesting and is still not widely studied in Indonesia at this time [2]. The improvement of soft soil by this method is considered very appropriate by considering the factor that soft soil has high water content, with heat induction it can accelerate the drainage rate of

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water in soft soil. So that the combination of Preloading and Thermal Induction can accelerate the rate of soil compression for infrastructure development purposes. Therefore, this research aims to determine the effect of Thermal Induction with variations in the inclusion of vertical and sloping types with preloading loading on soft soil at the radius where the Thermal Induction rod is embedded [3]. It is expected that from the results of this experimental study, empirical findings and an overall picture of soft soil behavior will be obtained, especially on the influence of soft soil mechanical properties, mineralogical aspects, soil thermo-mechanical [4].

2. Literature Review

2.1. Soft Soil Preloading

The preloading method is the initial loading with the weight of the embankment and equivalents around the weight of the structure used later. Soil conditions vary and are not always the same in each construction area, thus requiring precision in planning and implementing the construction itself. Soil strengthening (gained strength) that occurs due to the gradual preloading process or stage construction, changes in the level of saturation or wetting effect, creep strain, are factors that are often not taken into account in estimating consolidation decline [5]. Preloading aims to increase the shear strength in the soil, reduce the compressibility of the soil and prevent large settlements and possible damage to the building structure. Preloading is generally used on soils with low bearing capacities, such as soft clay soils and organic soils. This type of soil usually has the following characteristics: extreme moisture content, large compressibility, and small permeability coefficient [6, 7].

2.2. Thermal Induction in Clays

The relationship between thermal conductivity and soil properties such as mineral composition, dry bulk density (porosity), pore fluids, degree of saturation, moisture content, and temperature The geometric effect of soil particles on the thermal conductivity of saturated clay. Table 1 lists the thermal conductivity of the mineral (thermal conductivity) in the soil, which is one of the thermal parameters that shows the amount of energy that can be distributed in the soil [8].

<table>
<thead>
<tr>
<th>Types</th>
<th>Thermal Conductivity (W/m/K)</th>
<th>Thermal Capacity (J/Kg/K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>0.25 – 4.20</td>
<td>800 – 2.646</td>
</tr>
<tr>
<td>Sand</td>
<td>0.15 – 5.03</td>
<td>800 – 1.747</td>
</tr>
<tr>
<td>Gravel</td>
<td>4.44</td>
<td>1.175</td>
</tr>
</tbody>
</table>

2.3. Settlement Consolidations

A decrease in consolidation is a condition when the soil layer experiences additional loads, the pore water pressure will suddenly rise. The discharge of pore water goes along with a decrease in soil volume which causes a reduction in the soil layer. In clay soil with low probability, excess pore water stress takes a long time to dissipate, so that the decline in consolidation takes a very long duration. To calculate the consolidation decline using Equation 1 [9].

\[
S_c = \sum \left( \frac{C_c \times h}{1 + e} \times \log \left( \frac{\sigma_o + \Delta_o}{\sigma_o} \right) \right)
\]

Where: \( S_c \) = Settlement consolidation (m); \( C_c \) = Compression Index; \( H \) = High layer (m); \( e \) = Void; \( \sigma_o \) = Effective Pressure (t/m²); \( \Delta_o \) = Surcharge effective pressure (t/m²).

2.4. XRD (X-Ray Diffraction) Testing

The analysis method by X-ray diffraction (XRD) can be used to identify clay minerals because it emphasizes the crystal structure aspect of the mineral by referring to Bragg’s law and can be used to classify the type of mineral as long as the mineral has a certain crystal form even though its size is very small. Regarding the X-ray diffraction character of a single mineral which can guide the basic properties of the X-ray diffraction pattern for each type of clay mineral. Semi-quantitative XRD analysis was carried out to determine the proportion of montmorillonite, illite, and kaolinite or chlorite clay minerals which are useful for estimating the expansive properties of clay contained in rocks [10].
3. Material and Methods

3.1. Sampling Location

Soft soil for this study was obtained from Takalar, South Sulawesi, Indonesia. The sampling location is shown in Figure 1 below with coordinates 5°26’54.79”S 119°82’85.71” T. The research location is on the Takalar coast, a rice field irrigation area. The sample locations are shown in Figure 1.

Figure 1. Sampling location, Takalar, South Sulawesi, Indonesia

Initial condition partially due to the drier conditions associated with climate change. The soil thermal conductivity depends on the soil composition, bulk density and especially on the water content. Thus, moisture content might have an insulating effect, decreasing the temperatures reached on the soil after a heating event or might conduct heat pulses more efficiently, resulting in deeper penetration and, perhaps, higher temperature down the soil profile. Therefore, the impact of heating on the physical, chemical and biological soil properties depends, among other factors, on the soil moisture, which can affect both the direct impact of thermal shock and the further survival of soil microorganisms. Increasing heat conditions will make the soil stiff [11-13].

3.2. Model Test Procedure

Modelling the disturbed soil sample is placed in a test tube with the dimensions of the soil sample being tested in a test tube with a length of 150 cm, a width of 50 cm, and a height of 70 cm. The test tube is shown in Figure 3. Besides, the load given is 0.015 kg/cm², this loading is constant at each test temperature variation, where reconstitution is carried out with a mixing water and sediment pattern before thermal induction. The research method is to create a framework and stage flowchart as in Figure 2.
Observations are made at the radial zone (R1) or 15 cm from the thermal point as shown in Figure 3 which has a significant impact on the thermal effect. This observation is in the form of physical, chemical, and mechanical changes of the test sample [14, 15]. Physical test is carried out through a property test which includes specific gravity, grade, water, Atterberg limit. Mineralogical testing was carried out by SEM, EDS, and XRD tests to see changes before and after thermal application. Mechanical testing was carried out to determine changes in the values of shear strength, free compressive strength, and modulus of elasticity [16].
4. Results and Discussion

Testing the physical and mechanical characteristics of the soil was carried out to classify the type of soil used in the study. Based on the test results in the laboratory, the physical characteristics data were obtained which are shown in Table 2.

Table 2. Physical and mechanical characteristics of soft soil before and after heating

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Units</th>
<th>Test Results Initial Condition</th>
<th>Test Results After Heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity (Gs)</td>
<td>-</td>
<td>2.70</td>
<td>2.69</td>
</tr>
<tr>
<td>Initial water content (ω)</td>
<td>(%)</td>
<td>17.36</td>
<td>17.36</td>
</tr>
<tr>
<td>Soil unit weight</td>
<td>kN/m³</td>
<td>15.32</td>
<td>16.60</td>
</tr>
<tr>
<td>Dry soil unit weight</td>
<td>kN/m³</td>
<td>9.75</td>
<td>14.00</td>
</tr>
<tr>
<td>Degree saturation</td>
<td>(%)</td>
<td>90.32</td>
<td>51.59</td>
</tr>
<tr>
<td>Plasticity Index (PI)</td>
<td>(%)</td>
<td>36.60</td>
<td>34.30</td>
</tr>
<tr>
<td>Gravel fraction</td>
<td>(%)</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Sand fraction</td>
<td>(%)</td>
<td>5.20</td>
<td>5.20</td>
</tr>
<tr>
<td>Silt fraction</td>
<td>(%)</td>
<td>41.36</td>
<td>41.36</td>
</tr>
<tr>
<td>Clay fraction</td>
<td>(%)</td>
<td>53.44</td>
<td>53.44</td>
</tr>
<tr>
<td>USCS</td>
<td>CH</td>
<td>CH</td>
<td></td>
</tr>
<tr>
<td>Unconfined compressive strength</td>
<td>kN/m²</td>
<td>Very soft</td>
<td>67.22</td>
</tr>
<tr>
<td>Elasticity modulus</td>
<td>kN/m²</td>
<td>Very soft</td>
<td>4593.00</td>
</tr>
<tr>
<td>Cohesion</td>
<td>kN/m²</td>
<td>Very soft</td>
<td>21.75</td>
</tr>
<tr>
<td>Internal Friction Angle</td>
<td>Degree (°)</td>
<td>Very soft</td>
<td>23.00</td>
</tr>
<tr>
<td>Permeability</td>
<td>cm/second</td>
<td>0.000068</td>
<td>0.000015</td>
</tr>
</tbody>
</table>

Soil consistency shows the resistance to cohesion or adhesion of soil grains with other objects. This is indicated by the resistance of the soil to forces that will change its character [17, 18]. Soil conditions that are often a constraint and are relatively common are the soil that has non-uniform swelling and shrinkage properties as well as fairly high soil plasticity. The plasticity index is an important parameter as a measure of soil stability as a subgrade. Figure 4 shows that the consistency of illite clay soil.
The XRD (X-Ray Diffraction) results as shown in Figure 5 show the type of illite clay minerals, with X-Ray Diffraction (XRD), Chemical Composition, and Surface Morphology knew with an integrated system of Energy Dispersive Spectroscopy (EDS) and Polycrystalline Scanning in the Hexagonal system with the following lattice parameters. Substituted in the illite equation: $H_2KAl_3O_12 = Al_2O_34SiO_2H_2O + H_2O$.

The geometric effect of soil particles on the thermal conductivity value of clay can experience soil properties such as mineral composition, dry bulk density (porosity), fluid-pore, degree of saturation, moisture content, and temperature.
Identification of soil characteristics after the effect of thermal induction with the addition of free compressive strength ($q_u$). The unconfined compression test is one of the most common soil tests performed on clay soils. From the results of this test, it will be known that the failure stress parameter ($q_u$) is shown in Figure 7.

Thermal addition to the soil can cause the soil layer underneath to experience compression, resulting in deformation of soil particles, particle relocation, discharge of water or air in the pores, settlement caused by loading. The results of this test model show that in Figure 7 there is a decrease due to the addition of temperature with static loading. The prediction of a decline in the Asaoka method occurs as Figure 8.
The increase in the modulus of elasticity was due to the addition of the thermal induction value. The combination of these values tended to be directly proportional to the results of the free compressive strength test. The relationship between the unconfined compression test and temperature values is as shown in Figure 9.

5. Conclusion

A significant reduction in water content occurs in the radius closest to the heater (R1) or 15 cm. The higher the temperature given, the water content will decrease. The farther from the heater, the effect of smaller the temperature with the intention the water content does not change significantly. The main observation point is a distance of 15 cm from the center of the induction. At the induction it tends to be 400 °C, the water content is 17.36% from the initial water content of 59.07%, the soil cohesion is 21.75 kN/m², the value of free compressive strength is 67.72 kN/m² and the highest modulus of elasticity is 4593 kN/m², the decrease is 5.13 cm. XRD, SEM, EDS results before heating showed mineralogy 0 (65.06%), Ca (13.30%), Na (3.64%), Mg (2.15%), Al (6.63%), Si (8.52%), Sn (0.70%) and did not change significantly after heating at 400 °C. The results after heating included 0 (58.39%), Ca (14.09%), Na (0.72%), Mg (1.16%), Al (6.63%), Si (14.72%), Sn (2.54%). The novelty obtained is to change very soft conditions became medium conditions.

Figure 9. Graph of settlement all radial zone areas

Figure 10. Graph temperature and modulus of elasticity from the results of the free compressive strength test
6. Declarations

6.1. Data Availability Statement

The data presented in this study are available on request from the corresponding author.

6.2. Funding and Acknowledgements

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6.3. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Implementation of a Degassing System at the MSW Landfill

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Abstract
Aitolo-Akarnania prefecture, western Greece, is an area with strong earthquakes and large active fault systems. The most, the second half of the 20th century was characterized for the world community by the aggravation of the environmental problem. Anthropogenic pollution of the environment with the growth of industrial and agricultural production, the growth of cities, the size of the population, the volume of their consumption clearly indicates that the world community is on the brink of an abyss. The destruction of forests, pollution of water bodies, degradation of soil, flora and fauna, the emergence of new diseases clearly shows that if urgent and drastic measures are not taken to save the environment, the life of future generations is problematic. In Russia, as in other countries of the world, the amount of solid household waste has been sharply increasing lately. Therefore, their processing and disposal is becoming an increasingly urgent problem that requires the adoption of complex solutions. At the same time, overcrowded and smoking landfills, as well as formed unauthorized landfills are the main sources of environmental pollution. Landfills of solid municipal waste not only cause an epidemiological hazard, but due to the anaerobic decomposition of organic waste, causing the formation of explosive biogas, become a powerful source of biological pollution. Biogas generated at MSW landfills in the process of decomposition has a toxic effect on living organisms, contributes to the outbreak of fires, and is a source of unpleasant odors. This problem must be solved by introducing a degassing system at municipal solid waste landfills already at the stage of their operation. The proposed degassing system at the MSW landfill is aimed at reducing the negative impact of biogas on the environment.

Keywords: Degassing; Landfill; Municipal Solid Waste; Biogas.

1. Introduction
A sharp increase in consumption throughout the world in recent decades has led to a significant increase in the formation of MSW, the bulk of which is currently taken to landfills [1]. Landfill burial leads to a variety of close and distant consequences, negative both for humans and for the entire environment as a whole, occurring immediately or after some time [2, 3]. Among these consequences, attention is drawn to the biochemical fermentation of MSW with the release of biogas into the atmosphere. As know, biogas consists of methane (up to 55%), carbon dioxide (up to 45%) and a number of other volatile substances. Biogas is explosive and fire hazardous, it has a depressing effect on plant development, can have a suffocating effect, and according to various estimates, it has 7-23 times more impact on global climate change than CO2 [4].

The modern urban waste management system that has developed in Russia can be represented in the form of a block diagram (Figure 1). Waste is centrally collected and transported to large urban complexes, where it is further

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processed by one or more of the methods indicated in the block diagram. But the most voluminous on a global scale today is the method of burial at special landfills [5, 6].

![Figure 1. A flowchart of a municipal waste management system](image)

A landfill is a natural or artificial earth quarry, reaching an area of several hectares with a depth of 10-20 m, located above dense waterproof layers of earth and having a multilayer protective screen made of artificial and natural materials [7]. Drainage pipes are laid along the entire perimeter of the landfill to bring the generated wastewater and landfill gas (biogas) to the surface. The landfill is equipped with a special fire extinguishing system. Drainage water must undergo special treatment, and the resulting biogas must be neutralized or disposed of in some way [5].

The problem of landfill gas utilization in Russia is very acute for one simple reason: we practically do not have equipped landfills described above, but we have unequipped landfills, often not isolated from aquifers, without drainage wastewater and biogas, sufficient compaction and fire extinguishing systems [8, 9]. One of these is the Velizhansky MSW landfill which was put into operation in August 2010. The occupied area is 30 hectares, and the design capacity is 333,545 m$^3$/year, 230 thousand tons/year.

Landfill gas (biogas) is formed as a result of the fermentation of organic constituents in waste in the landfill body during biochemical decomposition [10]. Along with gaseous decomposition products, gaseous constituents of depositions (for example, greenhouse gases) and water vapor (in a saturated state) are also formed [11]. The resulting gases and vapors form a wet gas mixture of variable composition [12]. The main components of this mixture are methane CH$_4$ and carbon dioxide CO$_2$ [13]. Due to its main constituents, as well as the presence of other hazardous components [14], the emission of landfill gas can have a harmful effect on the environment in the form of [15]:

- Danger of explosion, burning, smoke;
- Landfill reclamation obstacles;
- Spreading of a certain odor;
- Release of toxic or hazardous constituents;
- Harmful influence on the climate.

Proceeding from this, gases must be collected and managed (treated) [16]. To date, there is no biogas removal system at the landfill. Fires resulting from the ignition of methane are extinguished by filling the landfill surface with inert soil and sprinkling the waste with water. In the first case, there is a loss of the volume [17] of maps that could be used for filling with waste, in the second - excessive moisture, leading to an acceleration of the fermentation processes (an increase in the yield of methane) and the formation of excess filtrate.

2. Materials and Methods

In order to determine the risks of changes in the volumes of biogas yield as a result of changes in the composition of the imported waste, calculations of the daily emission of components were carried out for different proportions of the content of active organic matter (Figure 2).
Figure 2. Predictive assessment of the yield of biogas components with different organic content
The maximum biogas yield in the most probable range of the content of the organic part of the waste (25-45%) will vary:

- Biogas in general - from 30,626 to 59,718 m$^3$/day;
- Methane - from 13,260 to 25,856 m$^3$/day;
- Carbon dioxide - from 17,366 to 33,861 m$^3$/day.

It is believed that the need for an active degassing system providing for the installation of gas-gathering wells (drains) and a technological system for biogas management (treatment) arises when more than 500 m$^3$/h methane is released [18]. Calculations showed that for the conditions of the Velizhansky MSW landfill, taking into account volume fluctuations due to changes in air temperature and morphological composition of waste, the following maximum values of biogas yield should be taken:

- Biogas in general - 1882 m$^3$/h (±606 m$^3$/h);
- Methane – 815 m$^3$/day (±262 m$^3$/day);
- Carbon dioxide – 1067 m$^3$/day (±344 m$^3$/day).

3. Results and Discussion

Thus, the results of the calculations confirm the need to implement a degassing system that allows the disposal of landfill gas at the stage of landfill operation [19]. The landfill gas management system is as follows. All horizontal drains are tied into a single gas-collecting header, which diverts the total gas volume to the management (treatment) system. Ensuring the disposal of landfill gas envisages collect and burn it in a high-temperature flare.

The gas treatment scheme includes:

- Removal of mechanical impurities on coarse and fine filter units;
- Removal of droplet moisture on the separator of the first separation stage (upstream of the compressor station);
- Increase in gas pressure by means of gas blowers installed at the compressor station;
- Biogas combustion in a flare.

With the help of a gas blower, which simultaneously creates both the vacuum ensuring the supply of gas from the landfill and the required yield pressure necessary for further supply through the pipeline systems, the gas is delivered to the gas combustion plant located in the eastern part of the landfill. Landfill gas that has passed the combustion process in a gas combustion plant loses unpleasant odors and is completely neutralized [20].

The combustion system consists of 2 main elements at high temperature flare and a container with the main equipment and elements that require protection from weather conditions. Biogas is supplied to the container. An indicative thermometer, indicative manometer, and temperature and pressure sensors are installed on the gas line inside the container, from which a signal is sent to the monitoring and control system. The landfill gas then enters a stainless-steel filter used to prepare the gas for compression. Excess moisture is discharged from the filter into a condensate collection tank located outside.

Following the filter, an electric valve is installed on the gas line, which cuts off the gas blower. The gas blower provides minimum inlet vacuum and outlet overpressure to feed the high-temperature flare. The blower capacity can be varied by means of frequency control (an inverter is included in the scope of delivery). Following the gas blowers at the outlet from the container, an indicative thermometer, an indicative pressure gauge, and temperature and pressure sensors are installed on the gas line.

The container houses the control system in the control compartment which ensures stable gas combustion, warns of errors, and turns off the system in case of an accident. The control system is supplemented with an optional EWON module (for remote monitoring and control) and a gas analysis system (for monitoring four gases). The container is heated and protects the equipment from precipitation and low temperatures. A flow sensor is installed outside the container the signal from which also comes to the control system. After that, the gas goes directly to the high-temperature flare for combustion. The gas combustion system is fully automated and starts/stops automatically. The high-temperature flash cycle starts as soon as an external start command is issued.

Combustion occurs at temperatures of at least 850 °C and with a holding time of 0.3 sec. This ensures complete combustion of methane, which complies with Russian and international norms and standards. This allows achieving complete combustion and significantly reducing harmful emissions and the formation of unpleasant odors [21].

The technical characteristics of the equipment used in the project are presented below (Figure 3).
To ensure reliable operation of the compressor unit, the compressor units are mounted in a block container. The block-container is a room for placing compressor equipment and automation systems in the corresponding compartments, ensuring a certain microclimate in them, reducing the sound pressure level.

The block container mounted air extraction from the upper zone through the baffle, providing a single breathability. Exhaust ventilation is switched on automatically from gas analyzers upon reaching 10% LEL (lower concentration limit of distribution) for methane. Air intake is from the upper zone. Exhaust ventilation can be switched on manually from a button located outside the front door to the BCU room for preliminary ventilation. The fans are located inside the container block. An emergency ventilation system is used to remove gases and smoke from the block container after the automatic powder fire extinguishing installation. The main parameters of the biogas combustion system are presented in Table 1.

Table 1. Parameters of the biogas combustion system

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flare unit brand</td>
<td>UFU-01.UHL.02.Z</td>
</tr>
<tr>
<td>Nominal capacity, m³ / h</td>
<td>1900</td>
</tr>
<tr>
<td>Maximum productivity, m³ / h</td>
<td>2200</td>
</tr>
<tr>
<td>Performance adjustment, %</td>
<td>20 - 100</td>
</tr>
<tr>
<td>Flare height, m</td>
<td>15</td>
</tr>
<tr>
<td>Internal diameter, mm.</td>
<td>2190</td>
</tr>
<tr>
<td>Outer diameter, mm.</td>
<td>2420</td>
</tr>
<tr>
<td>Installation weight, t</td>
<td>15</td>
</tr>
<tr>
<td>Block compressor installation</td>
<td>2VGS-32 / 0.88-1.18 PCh U1</td>
</tr>
<tr>
<td>Compressor</td>
<td>1 worker + 1 standby</td>
</tr>
<tr>
<td>Compressor power consumption, kw</td>
<td>37</td>
</tr>
<tr>
<td>Rotation frequency of the main rotor of the compressor, rpm</td>
<td>3000</td>
</tr>
<tr>
<td>Suction pressure, mbar</td>
<td>-100</td>
</tr>
<tr>
<td>Discharge pressure, mbar</td>
<td>60</td>
</tr>
<tr>
<td>Overall dimensions of the block container, mm:</td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>9000</td>
</tr>
<tr>
<td>Width</td>
<td>3000</td>
</tr>
<tr>
<td>Height</td>
<td>3800</td>
</tr>
<tr>
<td>Dry installation weight, t</td>
<td>15</td>
</tr>
</tbody>
</table>
4. Conclusions

The flare unit is designed for a maximum capacity of 1900 m³/h. The ventilation system starts to operate in normal operation conditions for a short period of time (for example, 5 minutes every hour) when a biogas leak is detected. Thus, the construction of a landfill gas utilization complex will minimize its negative local and global impact, as well as significantly improve the environmental situation in the region, while guaranteeing:

- Reduction of methane emissions to zero level, thanks to the controlled combustion process;
- Absence of emissions of hydrogen sulfide, mercaptans and other odorous gases that are harmful to human health;
- Improving the socio-economic status of the territory;
- Reducing the risk of explosions and fires in the territory [22, 23].

As mentioned above, landfill gas is a powerful factor in the negative impact of landfills on the environment. At the same time, according to the EPA, from one million tons of waste, the amount of gas is generated, sufficient to generate 0.78 MW of electricity, which is more than enough to clean the leachate of the landfill of these wastes.

5. Declarations

5.1. Data Availability Statement

The data presented in this study are available on request from the corresponding author.

5.2. Funding

The author received no financial support for the research, authorship, and/or publication of this article.

5.3. Conflicts of Interest

The author declare no conflict of interest.

6. References


Assessment and Evaluation of Blended Cement Using Bamboo Leaf Ash BLASH Against Corrosion

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Abstract

Concrete provides a high degree of protection against corrosion of embedded steel reinforcement. Owing to the harsh environmental conditions and the presence of aggressive elements from the marine environment, deteriorating corrosion affects the durability of reinforced concrete structures. This study evaluated the effectiveness of bamboo leaf ash BLASH as a supplementary cementing material or admixture with Portland cement to improve the durability of reinforced concrete structures. Specimens of 0, 10, 15, and 20% BLASH mixtures were prepared using 16, 20, and 25 mm Ø steel reinforcements. A total of 100 cylindrical specimens were cast and used in this study. The specimens were accelerated by corrosion using impressed current techniques and a galvanostatic method in a simulated environment. The results show that specimens with a BLASH content of 10% exhibited superior performance and exhibited longer corrosion initiation and propagation times. It has a higher resistance to acid penetration and lower corrosion rates. The crack parameters of the specimen with BLASH admixtures, such as the crack width and crack frequency, were negligible. The use of BLASH as an admixture strengthens its durability and improves its residual strength and serviceability.

Keywords: Bamboo-Leaf Ash; Admixtures; Crack Parameters; Pull-out Strength; Compressive Strength; Splitting Tensile Strength.

1. Introduction

Concrete is a composite material consisting of a binding medium within which aggregates are embedded [1]. Concrete, which has high alkalinity, initially provides a highly alkaline environment for composite materials [2]. Alkaline is used to form a passive film on a steel surface [3]. It provides a high degree of protection against the deterioration of the embedded steel reinforcement [4]. The embedded deformed steel reinforcement is protected to maintain its durability, ensure its safety [5], and function effectively [6]. However, one limitation of concrete, even of good quality, is the presence of microcracks, voids, and capillary pores, where deteriorating agents such as chloride can easily penetrate the structures [1], which in turn causes the loss of the protective characteristics of concrete [6, 7]. In addition to poor concrete quality and poor design and construction, owing to harsh environmental conditions, the deterioration of reinforced concrete structures occurs [1, 2]. Thus, it is noted that the interaction between the type of material and environmental exposure affects the deterioration of all structures [8]. Among the deterioration, corrosion of steel reinforcement due to chloride intrusion in concrete structures is the root cause of the deterioration problem of most structures located in coastal areas [2, 9].

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http://dx.doi.org/10.28991/cej-2021-03091707 © 2021 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
Corrosion of steel reinforcement, particularly in coastal areas, is a severe problem, and its prediction of occurrence is essential for structural and material engineers [10]. The deterioration due to corrosion of steel reinforcements significantly affects the durability of concrete structures [11]. When the corrosion of embedded steel reinforcement is initiated, it is difficult to stop its propagation, which affects its safety, loading capacity, and service life [12]. Corrosion causes a reduction in the steel bar area and bond strength between steel and concrete [13, 14], which leads to cracking, spalling, and delamination of the concrete cover [5]. According to Ademola and Buari (2014), corrosion has a detrimental effect on its structural performance, which significantly reduces the lifespan of structures [15]. Corrosion reduces the residual capacity of steel reinforcement, which affects the extent of residual serviceability of structures [16, 17].

On the other hand, cement, a basic component of concrete, is widely accepted as a common construction material because of the availability and abundance of raw materials, low costs, and adaptability of concrete to various shapes [13, 18-20]. However, the production of Portland cement is considered a major contributor to greenhouse gas emissions [21]. It releases about one ton of CO$_2$ emitted into the atmosphere [22], which is a major cause of global warming [13, 15].

Thus, researchers are currently on the challenge of using waste materials with pozzolanic properties in concrete production as cement replacement materials [23]. The selection of effective corrosion protection materials and methods must consider the environmental impacts as environmentally friendly, cost-effective, and compatible with concrete materials [10, 22]. Furthermore, to minimize the deterioration due to corrosion, the permeability and adequate concrete cover should be considered [6, 11].

Corrosion inhibitors, also known as barriers [3], help to prevent or minimize the corrosion of reinforced concrete structures. These admixtures can slow down or prevent the corrosion of the reinforcing steel in concrete. It was first investigated in the 1960s [7]. These alternative materials are generally selected based on the additional functionality that they offer and their cost-effectiveness [24]. Note that the protection mechanism of corrosion-inhibiting admixtures is already an integral part of the concrete matrix, thus directly reducing the permeability of the concrete [3].

Commonly used corrosion inhibitors, which are admixtures of cement include fly ash, slag cement formerly called ground granulated blast furnace slag, silica fume, rice husk, eggshell, and areca nut husk ash [19, 25]. Fly ash admixtures are one of the most well-known corrosion inhibitors that are popularly known. It exhibited better corrosion resistance than ordinary Portland cement concrete [26]. The higher resistivity of the fly ash admixture is evident from the reduced level of corrosion with fly ash concrete [12, 14].

In some countries, such as the Philippines, a significant amount of bamboo is processed, generating high volumes of solid waste. According to Thomas et al., bamboo is the highest-yielding and abundant construction material that can be found in any locality. It has an estimated annual production of 20 million tons worldwide [20]. It has proven to be an agro-waste product that is highly useful in the construction industry [27]. In addition, according to Kumar and Vasugi (2020), bamboo is best suited for use in construction because of its mechanical and chemical properties [28]. Apart from bamboo residues that are used as fibers, bamboo leaves are often burned in open landfills, negatively impacting the environment [23]. Bamboo leaf ash is a good pozzolanic material that reacts with calcium hydroxide to form calcium silicate hydrate [29, 30].

A mixture of Portland cement and bamboo leaf ash is known as “blended cement” or “composite cement [31]. Several studies have been performed to evaluate the characteristics of bamboo leaf ash in the partial replacement of Portland cement in construction. Akhtar and Sarmah (2018) investigated the physical and mechanical properties of partially replaced bamboo ash cement mortars. The compressive strength increases with curing time and decreases with increasing amounts of additives. A 10% bamboo leaf ash admixture was found to be optimum with better durability performance [19]. Kumar and Vasugi (2020) also reviewed and examined the physical and mechanical properties of bamboo-reinforced structural elements with geopolymer concrete. The bamboo material in geopolymer concrete enhanced its strength and proved its suitability for sustainable construction [28].

Frias et al. (2012) conducted a detailed scientific study of Brazilian bamboo leaf ash using different techniques such as XRF, XRD, SEM/EDX, FT-IR, and TG/DTG. The results of the study reveal that bamboo leaf ash cement with 10% and 20% yields better behavior with good physical and mechanical requirements for the existing European standards [32]. Ademola and Buari (2014) also examined the durability of bamboo leaf ash blended with cement in a sulfate environment. Blended cement with bamboo leaf ash has been proven to be resistant to magnesium sulfate, sodium sulfate, and calcium sulfate media, but it was found to perform better in soils rather than in seawater facilities [15]. Zea Escamilla and Habert (2014) prepared several percentages by weight of bamboo leaf ash to assess its strength and durability as a partial replacement in concrete. The 10% bamboo leaf ash had better results than the other bamboo leaf ash admixtures [23]. Dhinakaran and Chandana (2016) evaluated the compressive strength, pozzolanic activity, sorptivity, and porosity of blended calcined bamboo leaf ash. Fifteen percent of bamboo leaf ash was found to be useful as a replacement for cement, but with little compromise in its strength and durability characteristics. Their
work also verified that there is a substantial cost reduction in the replacement of cement with bamboo leaf ash. The cost incurred for cement was reduced by approximately 15% of its original value [33]. However, Dwivedi et al. (2006) reported that 20% bamboo leaf ash admixtures have a compressive strength comparable to that of ordinary Portland cement even after 28 days of curing [29].

Bannaravuri and Birru (2018) introduced bamboo leaf ash as an effective admixture for the development of aluminum alloy-based composites with 0, 2, 4, and 6% BLA content. The density of the fabricated composites decreased with the addition of bamboo leaf ash particles, and the hardness and tensile strength improved significantly [34]. Moraes et al. (2019) characterized bamboo leaf ash production with auto-combustion based on its pozzolanic behavior. Bamboo leaf ash admixtures are classified as highly conductive and have high pH values [35]. Onikeku et al. (2019) also examined the use of bamboo leaf ash as a supplementary cementitious material. BLA improved the split tensile, compressive, and flexural strength benchmarks by 10% as the optimum reinforcement level. BLA can be considered a good pozzolanic material that can reduce the cost of construction and improve concrete properties [36]. In addition, Kolawole et al. (2021) evaluated bamboo leaves and clay brick waste for their potential to serve as sustainable pathways for cementitious composites. The 10% BLA mixture helps achieve acceptable strength and resistance to chemical attacks [37].

Several studies have also been conducted to evaluate the effectiveness of other by-product waste agro-materials against corrosion deterioration. Chowdhury et al. (2015) evaluated the suitability of wood ash as a partial replacement for conventional concrete. The results showed that the strength of the concrete blended mixtures decreased with increasing wood ash content, but the strength increased with increasing age [16]. Kouloumbi and Batís investigated the corrosion resistance of mortar with Greek fly ash immersed in a 3.50% sodium chloride solution. This improved the corrosion behavior of the steel reinforcing bars.

The results indicated that a mixture of fly ash improved the corrosion behavior of the reinforcing steel bars [26]. In the work of Montemor et al. (2000), the corrosion behavior of blended cement with fly ash, which is up to 50% of the total binder, was tested. The addition of fly ash generally leads to an increase in the corrosion initiation time and a decrease in its corrosion rate [38]. Alaneme et al. (2014) investigated the corrosion and wear behaviors of Al matrix hybrid composites with agro-waste products such as bagasse, rice hush, bamboo leaves, groundnut, and coconut shell, in a 3.50% chloride medium. Electrochemical studies and wear tests were performed, and it was verified that 4% weight of bamboo leaf ash was observed to have a higher resistance than the other composite samples produced [18]. Gurdíán et al. (2014) evaluated industrial by-products mixed with cement as recycled aggregates. The results show that there are no significant differences in the resistance of such mixtures to natural chloride attacks. However, considering the carbonation attack, none of the mixtures exhibited highly aggressive conditions and had higher corrosion values than ordinary Portland cement [39]. Andrade et al. (1991) investigated the corrosion resistance of different types of blended cement against the ingress of chloride from natural Mediterranean Sea water using a polarization resistance technique for a period of five years. Blended cement lengths the initiation period of corrosion compared to plain concrete and has a lower corrosion rate [31]. In a study of Sharp et al. (2014), the durability of concrete mixtures with supplementary cementitious material SCMs was evaluated. The results verified that SCMs reduce the permeability and absorption of concrete and increase the durability of the structures [4]. From the work of Brühwiler and Mivelaz (1999), chloride ingress under climatic conditions and the effect of early cracks were examined. It was concluded that permeability has an overall control over the quality of the cover concrete of new structures [8]. Bheel et al. (2020) studied wheat straw ash as an admixture with cement. The strength of the concrete decreased as the percentage of WSA increased. The 10% replacement was comparable to that of conventional concrete. A decrease in strength was noted for 20 and 30% replacement of cement by WSA [21]. Abdul et al. (2019) studied the strength of wheat straw ash admixtures. It reduces the compressive strengths and mechanical properties of mixtures; however, flexural and indirect tensile strengths are significantly enhanced [40].

Although there are relatively few studies on the technological use of bamboo leaf ash reported in the literature, only a few studies have been conducted to evaluate the effectiveness of bamboo leaf ash as a corrosion inhibitor of reinforced concrete structures, particularly those exposed to coastal areas. This experimental study aimed to assess the effectiveness of BLASH in establishing mixtures with high durability and high resistance to corrosion, which helps to lengthen the design life and serviceability of structures exposed to a severely aggressive environment.

This study was conducted using the experimental procedures. The galvanostatic method using the impressed current technique was used to accelerate corrosion in an artificially controlled environment. Several measurements of the data were conducted after a period of accelerated corrosion. Crack parameters such as crack widths and crack frequency, corrosion variables such as corrosion rates, mass loss rate and corrosion levels, bond strength, durability, and residual strength were evaluated. The effectiveness of bamboo leaf ash BLASH against corrosion deterioration of structures exposed to excessive marine environments.
2. Materials and Methods

2.1. Research Flow Chart

The flow charts for the research methodology are as follows in Figure 1:

![Figure 1. Research flow chart]

2.2. Calcination of Bamboo Leaves

Bamboo leaves from Bayog species were collected during the leaf ash preparation. Figure 2 shows a photograph of the bamboo leaves. The bamboo leaves were dried in the sun in open air for complete combustion. The ash was allowed to cool before removal after burning. The ash of the bamboo leaves was kept in a well-sealed plastic or polythene to avoid moisture absorption. The dried sample was kept immediately to avoid attacks by environmental elements, which might reduce its strength. After calcination, the ashes were ground and sieved to below 90µm, fineness similar to that of Portland cement.

![Figure 2. Photograph of Bamboo leaf]

2.3. Particle Size Distribution

Sieve analysis was carried out on the bamboo leaf ash. Table 1 shows the particle size distribution of the bamboo leaf ash. Sieving analysis was also performed for fine and coarse aggregate materials that were used in the experiments for characterization. The aggregates used were free of organic matter. Figure 3 shows the gradation curves of the bamboo leaf ash.
Table 1. Sieve Analysis of Bamboo Leaf Ash

<table>
<thead>
<tr>
<th>Sieve size (inch)</th>
<th>Sieve size (mm)</th>
<th>Weight Retained (g)</th>
<th>Weight Passing (g)</th>
<th>Cumulative % of Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 4</td>
<td>4.74</td>
<td>9</td>
<td>1.8</td>
<td>98.20</td>
</tr>
<tr>
<td>No. 8</td>
<td>2.36</td>
<td>17.8</td>
<td>3.60</td>
<td>94.60</td>
</tr>
<tr>
<td>No. 16</td>
<td>1.18</td>
<td>55.5</td>
<td>11.1</td>
<td>83.50</td>
</tr>
<tr>
<td>No. 30</td>
<td>0.60</td>
<td>129.8</td>
<td>26</td>
<td>57.50</td>
</tr>
<tr>
<td>No. 50</td>
<td>0.300</td>
<td>100.2</td>
<td>20</td>
<td>23.20</td>
</tr>
<tr>
<td>No. 100</td>
<td>0.150</td>
<td>93.60</td>
<td>18.7</td>
<td>4.50</td>
</tr>
<tr>
<td>No. 200</td>
<td>0.075</td>
<td>10.10</td>
<td>2.0</td>
<td>2.50</td>
</tr>
</tbody>
</table>

Figure 3. Gradation Curve

2.4. Specimen Preparation

After calcination, ash from the Bayog leaf was used as an admixture. The percentage by weight of the mixture of bamboo leaf ash with cement was 0% for the control specimen and 10, 15, and 20%. The material mixtures were designed according to the ACI-211 mix proportion for the Class AA mixture. The water-to-cement ratio is 0.45. The ratio of 0.45, based on the maximum permissible water-to-cement ratio for concrete exposed to severe exposure to seawater. Table 2 lists the designed mixtures of materials used in the experiments.

Table 2. Designed Mixtures of Materials

<table>
<thead>
<tr>
<th>Cement-BLA mixture</th>
<th>Sand</th>
<th>Gravel</th>
<th>Water / Cement ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.90</td>
<td>2.14</td>
<td>0.45</td>
</tr>
</tbody>
</table>

A cylindrical specimen with a diameter of 150 mm and height of 300 mm was cast. Table 3 shows the specimen description with the corresponding percentage of bamboo leaf ash BLASH and cement for each specimen preparation. A total of 36 cylindrical specimens were prepared and subjected to an experimental study. Freshly mixed concrete with bamboo leaf ash content was scooped into a mold. Each mold was filled in three layers with concrete, and each layer was rammed 25 times with a tamping rod. All specimens were cured in water for hydration periods of 14 and 28 day.

In this work, the initial curing conditions that affect the chloride resistance of concrete were carefully performed in a curing tank with samples completely immersed in the curing media for the target hydration periods. The curing conditions of concrete influence the hydration process.
Table 3. Description of Specimen for Corrosion Acceleration

<table>
<thead>
<tr>
<th>Specimen ID No.</th>
<th>BLA Content (%)</th>
<th>Dimensions (mm)</th>
<th>Reinforcement Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-1</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I-2</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I-3</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I-4</td>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>II-1</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>II-2</td>
<td>10</td>
<td>150 diameter – 300 height</td>
<td>20</td>
</tr>
<tr>
<td>II-3</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>II-4</td>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III-1</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III-2</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III-3</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III-4</td>
<td>20</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Specimens were grouped with three (3) different diameters of the steel reinforcement: 16 mm, 20 mm, and 25 mm deformed rebars. Each reinforcement, without confinement, was placed in the midst of a cylindrical specimen, as shown in Figure 4, a detail of cast specimens. After 24 h, the cast specimens were de-molded and immersed in potable water for curing. All specimens were subjected to water immersion curing for 28 days, based on ASTM C192. Figure 5 shows a schematic of the cylindrical specimen. All 12 specimens were prepared and subjected to accelerated corrosion.

Figure 4. Detail of Specimen

Figure 5. Scheme of a Concrete Cylindrical Specimen
2.5. Accelerated Corrosion Program

The galvanostatic method was used to accelerate the reinforcement corrosion with a constant current density of 50 Ampere/cm². After 28 days of curing, the sample specimens were partially immersed in a 5% NaCl solution as a simulated environment for a period of 1 month. Every other day, the specimens were subjected to acceleration for 30 min. In the galvanostatic method, the wire is connected to the positive and negative poles. Figure 6 shows a scheme for accelerating the corrosion of cast specimens.

2.6. Compressive Strength Test

Specimens with different cement-bamboo leaf ash mixture contents were subjected to compressive strength tests. The descriptions are presented in Table 4. A total of 16 cylindrical specimens with a designed mixture of 28 MPa for 28 periods were cast. After 7 and 28 days of water curing, it was tested using a calibrated universal testing machine (UTM).

Table 4. Description of Specimen for Compressive Strength Test

<table>
<thead>
<tr>
<th>Designation of Samples</th>
<th>Dimension (mm)</th>
<th>Content</th>
<th>Age (Days)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cement (%)</td>
<td>BLA (%)</td>
</tr>
<tr>
<td>CT-A</td>
<td>150 diameter – 300 height</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>CT-B</td>
<td></td>
<td>90</td>
<td>10</td>
</tr>
<tr>
<td>CT-C</td>
<td></td>
<td>85</td>
<td>15</td>
</tr>
<tr>
<td>CT-D</td>
<td></td>
<td>80</td>
<td>20</td>
</tr>
</tbody>
</table>

2.7. Acid Resistance Test

After 28 days of curing in water, the cast specimen was immersed in a 5% sulfuric acid H₂SO₄ solution. All edges and surfaces were maintained without crack-up and immersed in water for a period of 10-days. Before the specimen was immersed in an acid solution, the weight of the specimen was measured. After 10 days, the cylinder was removed and weighed again to determine the weight loss due to acid penetration. The compressive strength of each specimen was determined before and after chemical testing. Descriptions are presented in Table 5.

Table 5. Description of Specimen for Acid Resistance Test

<table>
<thead>
<tr>
<th>Designation of Samples</th>
<th>Dimension (mm)</th>
<th>Content</th>
<th>QTY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cement (%)</td>
<td>BLA (%)</td>
</tr>
<tr>
<td>ART-A</td>
<td>150 diameter – 300 height</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>ART-B</td>
<td></td>
<td>90</td>
<td>10</td>
</tr>
<tr>
<td>ART-C</td>
<td></td>
<td>85</td>
<td>15</td>
</tr>
<tr>
<td>ART-D</td>
<td></td>
<td>80</td>
<td>20</td>
</tr>
</tbody>
</table>
2.8. Tensile Strength Test

After the specimen was subjected to an acceleration program, the concrete specimen was crushed and the steel reinforcement bar was removed. The steel reinforcement rebars were weighed before and after the acceleration. The steel reinforcement was subjected to tensile strength testing.

2.9. Calculating Parameters

The weights of the rebars before and after corrosion acceleration are tabulated. Area reduction was determined using the weight of the corroded bars.

\[ A_{cs} = \frac{W_t}{(L \times \gamma_{\text{iron}})} \]  

Where:
- \( A_{cs} \) = Actual area of corroded reinforcement bar (mm\(^2\));
- \( W_t \) = weight of reinforcement after corrosion, and rust removed (g);
- \( L \) = Length of the specimen (mm);
- \( \gamma_{\text{iron}} = 0.00785 \text{ g/mm}^3 \) (steel).

The actual mass of rust per unit surface area in accordance with ASTM G1 on rebars extracted from the concrete specimen after the accelerated corrosion test was computed as follows:

\[ M_{ac} = \frac{(W_i - W_f)}{\pi DL} \]

Where:
- \( M_{ac} \) = actual mass of rust per unit surface area of the bar (g/cm\(^2\));
- \( W_i \) = Initial weight of the bar before corrosion (g);
- \( W_f \) = Weight after corrosion (g) for a given duration of induced corrosion (t);
- \( D \) = Diameter of the rebar (cm);
- \( L \) = Length of the rebar sample (cm).

Rate of corrosion was determined using corrosion current density, \( i_{corr} \):

\[ i_{corr} = \frac{M_{ac} F}{EWt} \]

Where:
- \( i_{corr} \) = corrosion current density (µAmp/cm\(^2\));
- \( M_{ac} \) = actual mass of rust per unit surface area of the bar (g/cm\(^2\));
- \( F \) = Faraday’s constant (96487 Amp-sec);
- \( EW \) = equivalent weigth of steel (27.925 for steel);
- \( t \) = time of accelerating corrosion (sec).

The corrosion level was expressed using the equation:

\[ C_r = \frac{G_o - G}{g_o L} \times 100\% \]

Where:
- \( G_o \) = is the initial weight of the reinforcement before corrosion;
- \( G \) = is the weight of reinforcement after removal of the corrosion products;
- \( g_o \) = is the weight per unit length of the reinforcing bar;
- \( L \) = is the bond length.
2.10. Splitting Tensile Test

After curing the specimen for 7 and 28 d, the immersed specimens were removed and allowed to dry. The machine was set in the required range, diametrical lines, and dimensions of the specimen. The specimen was loaded continuously without shock at uniform rates until failure occurred, and the failure load was recorded.

The splitting tensile strength formula used was:

\[ T_{sp} = \frac{2P}{\pi DL} \] (5)

Where:
- \( T_{sp} \) = Splitting tensile strength
- \( P \) = Applied load
- \( D \) = Diameter of the specimen
- \( L \) = Length of the specimen

3. Results and Discussion

The following discusses the results of the various experiments conducted to evaluate and assess the effectiveness of bamboo leaf ash as a corrosion inhibitor admixture of cement.

3.1. Compressive Strength

Figure 7 shows that the curing time and percentage content of admixtures with bamboo leaf ash are the two primary factors that influence the compressive strength of the blended cement mixtures. When the percentage content of BLASH increased, the compressive strength of the blended cement mix decreased. However, at each BLASH percentage, the compressive strength increased with an increase in the curing period. As the curing time increased, the compressive strength of the samples also increased. At 28 d of curing, the compressive strength was higher compared with the other shortest curing periods (7, 14, and 21 days). However, although the compressive strength of blended cement with BLASH content increased with time, their values were lower than those of the control specimen (0% bamboo leaf ash).

![Figure 7. Correlation of BLASH content (%) with Compressive Strength](image)

As shown in Figure 7, the compressive strength of the blended cement mixtures decreased when the percentage of bamboo leaf ash increased. The compressive strength of the mixtures with BLASH content (10, 15, and 20%) was lower than that of the control mixture with 0% BLASH content. The compressive strength of blended cement with 10% BLASH reduced the compressive strength of the cement by 10.89% (0% BLASH). The compressive strength of
blended cement with 15% BLASH was reduced by approximately 17.22% compared to that of the control specimen. The compressive strength was reduced by 22.63% with 20% BLASH content in the mix compared to that of the control specimen.

As shown in Table 6, a mixture with a higher BLASH content has lower compressive strength; however, it increases in the latter days of curing, usually after 21 days of curing. From the same table, it was shown that a 10% BLASH content of mixtures provides a higher compressive strength after 28 days of curing, as compared with other BLASH contents (15 and 20%).

Based on the work of Dhinakaran and Chandana (2016), for 10, 15, and 20% blended cement with bamboo leaf ash, the reduction in compressive strength at 28 days was 18, 10, and 16%, respectively [33]. The reduction was significant with 10 and 20% mixtures of bamboo leaf ash admixtures, but it had only a 10% reduction in strength when using 15% bamboo ash leaf admixtures. Compared with the results obtained, 10% mixtures of bamboo leaf ash had a lower reduction in strength, with 9% only at 28 days curing period, compared to other mixtures of BLASH (15 and 20%). Furthermore, according to Dhinakaran and Chandana (2016), with 25 and 30% bamboo leaf ash, the reduction in strengths drastically increases [33]. According to a similar study by Zea Escamilla and Habert (2014), the compressive strength of cement blended with bamboo leaf ash with 5, 10, and 20% was less than that of the control mixtures by 11, 21, and 41%, respectively. The results of the experiments conducted by Zea Escamilla and Habert (2014) were approximately twice those obtained from actual experiments. In both cases, the compressive strength decreased with an increase in the bamboo leaf ash content [23]. Similar to the work of Kumar and Vasugi (2020), the compressive strengths of blended cement with 10 and 20% had minimal reductions of 1.20 and 6.70%, respectively. However, at 28 and 90 d of curing, the strength of the specimens was equal to that of the control mortar [28].

The increase in compressive strength with curing age and percentage of admixtures was confirmed by Akhtar and Sarmah (2018), when the percentage of bamboo increased from 5 to 25%, the compressive strength of the specimens decreased by 43.90 to 76% on the 7th day of curing, whereas during the 56th curing, it decreased by 20 to 70% compared to the control specimen with 0% bamboo leaf ash content [19]. In addition, similar to the findings of Ademola and Buari (2014), the compressive strength of the bamboo leaf ash-blended concrete up to 10% performed better and would be acceptable and considered as a good development for construction in any sulfate environment [15]. However, from the work of Frías et al. (2012), bamboo leaf ash blended with cement mortars showed compressive strength values that were practically equal to those of the control mortar after 28 days of curing [32]. According to Dwivedi et al. (2006), 20% bamboo leaf ash content has a compressive strength equivalent to that without ash [29]. Moreover, according to Moraes et al. (2019), all mortars with bamboo leaf ash content have the same strength as ordinary Portland cement, even after 90 days of curing [35]. However, compared to this study, 20% bamboo leaf ash had a greater reduction in strength compared to the control specimen (0% BLASH).

The reduction in compressive strength is due to incomplete hydration of the mixture of cement with BLASH at the ages of 7-14-21 days of curing. The later day’s strength increase of the mixtures with BLASH content is due to the pozzolanic reaction. Thus, more water is required for a higher percentage of BLASH content to achieve the required workability. The compressive strength at 28 d of hydration was comparable to that of the control specimen. As verified from this experimental study, 10% bamboo leaf ash-blended cement is acceptable and better than other percent content of bamboo leaf ash. Hence, concrete blended with bamboo significantly produces durable concrete mixtures, but with compromised strength. Thus, BLASH-blended cement is suitable for construction, but with a longer period of hydration or curing.

### 3.2. Acid Resistance

Concrete is susceptible to acid attack due to the high alkalinity of Portland cement concrete, which is why acid resistance is a desirable property for structural materials exposed in an aggressive marine environment, such as seawater. An acid attack causes the concrete to lose its strength and quickly deteriorates.

<table>
<thead>
<tr>
<th>Designation of Samples</th>
<th>BLA Content (%)</th>
<th>Compressive Strength (MPa) @ 7 days</th>
<th>@ 14 days</th>
<th>@ 21 days</th>
<th>@ 28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>CT-A</td>
<td>0</td>
<td>15.00</td>
<td>16.10</td>
<td>16.49</td>
<td>17.10</td>
</tr>
<tr>
<td>CT-B</td>
<td>10</td>
<td>13.11</td>
<td>14.13</td>
<td>14.89</td>
<td>15.56</td>
</tr>
<tr>
<td>CT-C</td>
<td>15</td>
<td>12.76</td>
<td>12.98</td>
<td>13.65</td>
<td>14.13</td>
</tr>
<tr>
<td>CT-D</td>
<td>20</td>
<td>12.13</td>
<td>12.27</td>
<td>12.65</td>
<td>12.94</td>
</tr>
</tbody>
</table>

As shown in Table 6, a mixture with a higher BLASH content has lower compressive strength; however, it increases in the latter days of curing, usually after 21 days of curing. From the same table, it was shown that 10% BLASH content of mixtures provides a higher compressive strength after 28 days of curing, as compared with other BLASH contents (15 and 20%).
The weight reduction of the blended cement mixtures is shown in Figure 8. The control specimen (0% BLASH), once exposed to an acidic environment, showed a 0.70% reduction in weight. BLASH with 10% content had a lower weight reduction of 0.50%, and it had the highest resistance to acid attack compared with the 15 and 20% BLASH contents. With 10% BLASH content, the weight reduction was reduced to 0.50%; however, with further addition of BLASH in the mixtures, the weight reduction increased exponentially. The replacement of concrete with 10% BLASH helped to produce a mixture that enhanced the permeability of the concrete, which sealed the specimen and prevented the intrusion of acid chemicals into the produced mixtures.

![Figure 8. Weight Reduction in Acid Resistant Test](image)

Table 7 lists the weight losses of the specimens during the acid resistance tests. The 10% BLASH content reduced the weight of the mixtures after the acid test was performed. Ten percent of BLASH mixtures signifies the right amount of admixtures with cement in order to produce structures highly resistant to the intrusion of calcium salts from marine water. This helps to lower the detrimental effects of all types of acids in structural concrete elements.

<table>
<thead>
<tr>
<th>Designation of Samples</th>
<th>BLA Content (%)</th>
<th>Weight in Before Test</th>
<th>Weight in After Test</th>
<th>Weight Reduction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ART-A</td>
<td>0</td>
<td>2.700</td>
<td>2.690</td>
<td>0.70</td>
</tr>
<tr>
<td>ART-B</td>
<td>10</td>
<td>2.675</td>
<td>2.630</td>
<td>0.50</td>
</tr>
<tr>
<td>ART-C</td>
<td>15</td>
<td>2.645</td>
<td>2.637</td>
<td>0.80</td>
</tr>
<tr>
<td>ART-D</td>
<td>20</td>
<td>2.750</td>
<td>2.733</td>
<td>1.70</td>
</tr>
</tbody>
</table>

Table 8. Compressive Strength after Acid Resistance Test

<table>
<thead>
<tr>
<th>Designation of Samples</th>
<th>BLA Content (%)</th>
<th>Compressive Strength (MPa)</th>
<th>Strength Reduction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Before Acid Test</td>
<td>After Acid Test</td>
</tr>
<tr>
<td>CT-A</td>
<td>0</td>
<td>17.10</td>
<td>15.30</td>
</tr>
<tr>
<td>CT-B</td>
<td>10</td>
<td>15.56</td>
<td>15.56</td>
</tr>
<tr>
<td>CT-C</td>
<td>15</td>
<td>14.13</td>
<td>12.20</td>
</tr>
<tr>
<td>CT-D</td>
<td>20</td>
<td>12.94</td>
<td>11.10</td>
</tr>
</tbody>
</table>

In Table 8, the compressive strengths of the samples with 0, 10, 15, and 20% bamboo leaf ash admixtures before and after the acid test are listed. As shown in Figure 9, a mixture with 10% BLASH content had a lower percent reduction in its compressive strength (0.03%) when it was exposed to acid chemicals. The 20% BLASH content has a higher percentage reduction of its compressive strength, which is 14.22%; 15% BLASH content has 13.66% reduction; and 0% BLASH a control specimen has a 10.53% reduction.
The results of this experimental study conformed to those of Andrade et al. (1991), who found that blended cement neutralizes sulfuric acid with 10% bamboo leaf ash from simulated marine water. However, with 15 and 20% bamboo leaf ash content, it triggers the corrosive property of sulfuric acid from the marine environment and reduces its compressive strength [31]. In addition, in a similar study by Zea Escamilla and Habert (2014), bamboo leaves blended with cement showed 0.60, 3, 0.50 and 2.20% reductions in strength for 0, 5, 10, and 15%, respectively [23]. Compared to the actual experiment conducted, a 10% BASH content had the same percent reduction in strength. Kolawole et al. confirmed the results of this study. Bamboo leaf ash with a 10% admixture exhibited the highest resistance against the attack of deleterious acids. Thus, the durability of the structures can be improved by using a 10% bamboo leaf ash admixture.

3.3. Corrosion Variables

3.3.1. Mass Loss Rate

As shown in Figure 10, the two parameters affect the mass-loss rate of the embedded steel reinforcement specimens, percentage content of bamboo leaf ash, and diameter of deformed steel reinforcements. A small reinforcement (16 mm) diameter has higher loss rates compared to the two other larger reinforcement diameters (20 and 25 mm). A smaller reinforcement diameter (16 mm) is less resistant to corrosion deterioration; thus, it has a higher mass loss rate.

In each group with different reinforcement diameters (16, 20, and 25 mm), BASH with 10% content had the lowest mass loss rates. When the amount of BASH content increased by more than 10%, the rate of mass loss was higher than that of the control specimen (0% BASH). Thus, when the percentage of BASH increased further, the mass loss rate increased exponentially.

The results of experiments conducted by Alaneme et al. (2014) confirmed that an increased percentage of bamboo leaf ash leads to a decrease in the corrosion resistance of the specimens [18]. However, an accurate amount of bamboo leaf ash (10%) developed a matrix of concrete that improved its resistance against the intrusion of deteriorating elements that could alter the diameter and weight of the embedded steel reinforcement. This helps prevent the penetration of deleterious products that cause corrosion of the embedded reinforcement steel.

Specimens that have a lower resistance against corrosion deterioration have higher corrosion rates, and they have a significant reduction in mass [17]. The reduction in the cross-sectional area of steel rebars was greater with 15% and 20% BASH content as compared with 10% content of bamboo leaf ash admixtures. Thus, the quality of the cement-blended admixtures was improved by 10% BASH content.

The results verified that the corrosion resistance of the mixtures decreased when the amount of bamboo leaf ash increased. Furthermore, the mass loss rate is directly correlated with the rate of corrosion of the specimen [16]. However, the corrosion rates of the specimens depend on the quality of the concrete mixtures that protect the embedded steel reinforcement specimens.

Figure 9. Compressive Strength Reduction in Acid Resistant Test
3.3.2. Corrosion Level

In Figure 11, the blended cement mixture with 10% BLASH content has a lower corrosion level. The control sample had a maximum corrosion level of 5.74%, and 10% BLASH had a maximum corrosion level of 4.92%. With a further increase in bamboo leaf ash, 15% BLASH had a maximum corrosion level of 6.99%, and 20% BLASH had a maximum corrosion level of 9.08%.

A mixture with 10% BLASH content had a lower corrosion level than the other BLASH contents (15 and 20%). Cement blended with 10% bamboo leaf ash developed a mixture that has a lower permeability because a mixture has a higher resistance against ingress of deleterious substances that cause corrosion deterioration of the embedded reinforcement steel. The 10% bamboo leaf ash admixtures enhanced the quality of mixtures of the blended cement by reducing its permeability, and it was evident that the lower corrosion levels were acquired for each specimen with different deformed steel reinforcement diameters. This reveals that a good quality mixture enhanced by using 10% bamboo leaf ash admixtures provides perfect protection for the embedded steel reinforcement owing to the reduced corrosion level of the specimens compared to other samples with a higher percentage of bamboo leaf ash.
Montemor et al. (2000) provided a criterion for determining the degree of corrosion. Table 9 presents an interpretation of the corrosion rates. Control specimens (0% BLASH) with corrosion rates from 0.00137-0.0035 mm/year were categorized as moderate in corrosion; 10% BLASH with corrosion rates from 0.00105-0.0030 mm/year, 15% BLASH with corrosion rates from 0.00169-0.00428 mm/year, and 20% BLASH with corrosion rates from 0.00217-0.00555 mm/year, were considered moderate corrosion. As shown in Table 9, a specimen with a higher corrosion rate can be predicted as being at more risk of its deterioration [38].

3.4. Crack Indices

3.4.1. Crack Width

As shown in Figure 12, the 10%-BLASH content had negligible crack widths in the 16, 20, and 25 mm diameter deformed steel reinforcements, respectively. 0%-BLASH has a wider crack width (0.31 mm), 15%-BLASH has 0.003 mm crack widths, and 20%-BLASH has 0.003 mm cracks.

A mixture of 10% bamboo leaf ash admixtures helped to limit the width of the cracks formed on the surface of the concrete. A mixture of 10% bamboo leaf ash enhanced the bonds of finer particles that constituted the blended concrete mix. It helps to resist the formation of hairline cracks, which could be a pathway for the deleterious elements to penetrate the concrete surface.

According to Dacuan and Abellana (2020), a wider crack width accelerates the ingress of deleterious substances into the concrete surface. This leads to a faster deterioration in the corrosion of steel reinforcements [17]. As shown in Figure 12, mixtures of cement with more than 10% bamboo leaf ash content are more prone to deterioration because they have wider crack widths.

Furthermore, according to Dacuan and Abellana (2021), the width of the crack due to corrosion has a linear correlation with the corrosion level. High corrosion levels lead to a significant amount of rust products that push the surface of the concrete, resulting in wider cracks on its surface [9].

![Figure 12. Correlation of BLASH content with Crack Width](image-url)
3.4.2. Crack Frequency

As shown in Figure 13, 0%-BLASH had the highest number of cracks frequency, with a maximum of 52 crack formations; 15%-BLASH has 20 cracks, and 20%-BLASH has 25 cracks. Cracks were formed using a 10% BLASH. A mixture of 10% bamboo leaf ash enhances the quality of the mixtures by which the formation of hairline cracks resists. Crack formation was not detected in the specimens with 10% BLASH. However, increasing the amount of bamboo leaf ash in concrete mixtures leads to the formation of fewer cracks on the concrete surface. As shown in Figure 13, a mixture of bamboo leaf ash prevents the formation of cracks. Bamboo leaf ash admixtures reduced the number of cracks formed on the concrete surface. The specimen with ordinary Portland cement has a significant number of cracks on the concrete surface, as shown in the same figure.

Although the diameter of the steel reinforcement affects the formation of cracks, it was verified that a mixture of blended cement with admixtures that covers up to protect the embedded steel reinforcement significantly affects the deterioration of reinforced concrete structures. A quiet consideration should be given to the quality of the mixtures used in the construction with cement admixtures. This reveals that an accurate percentage of bamboo leaf ash admixtures enhances the quality of the mixtures, which improves their permeability and pore spaces. The 10% bamboo leaf ash content significantly enhanced the concrete blended mixtures by reducing the crack formation on its surfaces.

![Figure 13. Correlation of BLASH content with Crack Frequency](image)

3.5. Bond Strength

The bond strength of concrete and steel reinforcement is influenced by the quality of the mixtures, and the sizes of the main reinforcement used. A larger reinforcement diameter (25 mm) has a higher bond strength than a smaller reinforcement diameter (16 and 20 mm).

As shown in Figure 14, a mixture with 10% BLASH content has a higher bond strength, which is consistent for the three groups of steel reinforcements of different sizes. The 10% BLASH content significantly enhanced the bond strength between the particles in the mixtures. The higher bonds that develop in the mixtures of blended cement help to resist the penetration of deleterious elements and protect the reinforcement steel against corrosion deterioration.

According to Dacuan and Abellana (2021), the corrosion of steel reinforcement leads to a reduction in the bond strength between the steel and concrete interface [9]. However, it is a blended mixture of concrete that provides significant protection to the steel reinforcement and maintains the bond strength between the surfaces of two materials, steel and concrete elements. Although steel, which has a higher corrosion resistance, is used when it is exposed to deleterious substances owing to the poor quality of mixtures, deterioration of concrete structures will still occur. Thus, a 10% bamboo leaf ash admixture helps to develop a strong bond between steel and concrete by reducing the permeability of the mixtures and prohibiting the ingress of chloride ions.
3.6. Durability

Corrosion formation leads to an alteration in the yield strength of the steel reinforcement rebars. The reduction in the steel cross-sectional area led to a reduction in the strength of the steel rebar. It was used to estimate the residual strength of steel reinforcement. A normalized cross-sectional area, which is the ratio of the corroded cross-sectional area to its original area, was used to determine the residual strength of the corroded reinforcement structures in terms of its rebar cross-sectional area.

\[ R.S = \frac{A_{st}}{A_0} \]  

(6)

Where: \( R.S \) = Residual strength; \( A_{st} \) = Area of the rebar at time \( t \) (years); \( A_0 \) = Area before corrosion (\( t_0=0 \)).

\[ A_{st} = \frac{n\pi}{4} [\phi - r_{corr}(t - t_0)]^2 \]

(7)

Where: \( \phi \) = Diameter of the rebar; \( n \) = Number of bars in a layer; \( r_{corr} \) = Rate of corrosion in terms of penetration rate (mm/year); \( t \) = Time of corrosion (years); \( t_0 \) = Corrosion initiation time.

### Table 8. Residual Strength of Corroded Rebars (16 mm-diameters)

<table>
<thead>
<tr>
<th>t (years)</th>
<th>BLA-0%</th>
<th>BLA-10%</th>
<th>BLA-15%</th>
<th>BLA-20%</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>10</td>
<td>99.68</td>
<td>99.75</td>
<td>99.65</td>
<td>99.56</td>
</tr>
<tr>
<td>30</td>
<td>99.06</td>
<td>99.24</td>
<td>98.96</td>
<td>98.68</td>
</tr>
<tr>
<td>40</td>
<td>98.74</td>
<td>98.99</td>
<td>98.62</td>
<td>98.24</td>
</tr>
<tr>
<td>50</td>
<td>98.43</td>
<td>98.74</td>
<td>98.27</td>
<td>97.81</td>
</tr>
<tr>
<td>60</td>
<td>98.11</td>
<td>98.49</td>
<td>97.93</td>
<td>97.37</td>
</tr>
<tr>
<td>70</td>
<td>97.80</td>
<td>98.23</td>
<td>97.59</td>
<td>96.94</td>
</tr>
<tr>
<td>80</td>
<td>97.49</td>
<td>97.98</td>
<td>97.24</td>
<td>96.50</td>
</tr>
<tr>
<td>90</td>
<td>97.18</td>
<td>97.73</td>
<td>96.90</td>
<td>96.07</td>
</tr>
<tr>
<td>100</td>
<td>96.87</td>
<td>97.48</td>
<td>96.56</td>
<td>95.64</td>
</tr>
</tbody>
</table>
Table 9. Residual Strength of Corroded Rebars (20 mm-diameters)

<table>
<thead>
<tr>
<th>t (years)</th>
<th>BLA-0%</th>
<th>BLA-10%</th>
<th>BLA-15%</th>
<th>BLA-20%</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>10</td>
<td>99.84</td>
<td>99.88</td>
<td>99.80</td>
<td>99.76</td>
</tr>
<tr>
<td>20</td>
<td>99.68</td>
<td>99.76</td>
<td>99.60</td>
<td>99.52</td>
</tr>
<tr>
<td>30</td>
<td>99.52</td>
<td>99.64</td>
<td>99.40</td>
<td>99.28</td>
</tr>
<tr>
<td>40</td>
<td>99.36</td>
<td>99.52</td>
<td>99.20</td>
<td>99.04</td>
</tr>
<tr>
<td>50</td>
<td>99.20</td>
<td>99.40</td>
<td>99.00</td>
<td>98.81</td>
</tr>
<tr>
<td>60</td>
<td>99.04</td>
<td>99.28</td>
<td>98.81</td>
<td>98.57</td>
</tr>
<tr>
<td>70</td>
<td>98.88</td>
<td>99.16</td>
<td>98.61</td>
<td>98.33</td>
</tr>
<tr>
<td>80</td>
<td>98.72</td>
<td>99.04</td>
<td>98.41</td>
<td>98.09</td>
</tr>
<tr>
<td>90</td>
<td>98.57</td>
<td>98.92</td>
<td>98.21</td>
<td>97.86</td>
</tr>
<tr>
<td>100</td>
<td>98.41</td>
<td>98.80</td>
<td>98.01</td>
<td>97.62</td>
</tr>
</tbody>
</table>

Figure 15. Influence of BLASH Content to Residual Strength (16 mm diameter)

Figure 16. Influence of BLASH Content to Residual Strength (20 mm diameter)
Tables 8, 9, and 10 list the residual strengths of the corroded rebar for deformed steel diameters of 16, 20, and 25 mm, respectively. Figures 15 to 17 illustrate the residual strengths of each deformed rebar with different percentage mixtures of bamboo leaf ash admixtures. It was verified from all figures that for all sizes of steel reinforcement diameters, 10% bamboo leaf ash admixtures have a lower reduction in their residual strength once they are afflicted with corrosion deterioration of the steel reinforcement used. The addition of 10% bamboo leaf ash admixtures significantly lengthened the time required for the initiation of corrosion and propagation.

The overall results indicate that the strength of mixtures blended with bamboo leaf ash increases gradually up to 10%, but there is a reduction in strength of 15 and 20% due to extension in the initial and final settings and a quick loss of workability. The results proved that a 10% replacement of cement by BLASH was the optimum percentage for obtaining maximum strength values.

The results were confirmed by Zea Escamilla and Habert (2014), durability significantly improved with 10% replacement of cement with bamboo leaf ash BLASH [23]. The durability of mixtures with a 10% bamboo ash admixture was enhanced by reducing the permeability of the mixtures blended with an accurate amount of admixtures, thus strengthening its resistance against the attack of deleterious elements such as acid and chloride ions. Generally, the replacement of ordinary Portland cement with bamboo leaf ash improves the physical and mechanical properties of mixtures, and the durability is enhanced.
4. Conclusions

The following were the conclusions:

- The compressive strength of the blended cement mixtures decreased as the percentage of bamboo leaf ash BLASH increased. However, as the curing time increased, the compressive strength of the samples also increased.
- The replacement of concrete with 10% BLASH helped to produce a mixture that enhanced the permeability of the concrete, which sealed the specimen and prevented the intrusion of acid chemicals into the mixtures.
- When the amount of BLASH content increased by more than 10%, the rate of mass loss was higher than that of the control specimen (0% BLASH).
- The 10%-BLASH had negligible crack widths, both in 16, 20, and 25 mm diameter steel reinforcements. 0%-BLASH has a wider crack width (0.31 mm), 15%-BLASH has 0.003 mm crack widths, and 20%-BLASH has 0.003 mm cracks.
- A larger reinforcement diameter (25 mm) has a higher bond strength than a smaller reinforcement diameter (16 and 20 mm). The quality of the mixtures significantly affected the bonding between cement and steel. The 10% BLASH content has a higher bond strength, which is consistent for the three groups of steel reinforcements of different sizes.
- The overall results of the mechanical strength properties indicate that the strength values increase gradually up to 10%, but for 15 and 20%, there is a reduction in strength values due to extension in initial and final settings and quick loss of workability. The results proved that a 10% replacement of cement by BLASH was the optimum percentage for obtaining maximum strength values.

5. Declarations

5.1. Author Contributions

All authors contributed to the study conception and design. Material preparation, data collection and analysis. All authors have read and agreed to the published version of the manuscript.

5.2. Data Availability Statement

The data presented in this study are available in article.

5.3. Funding

The study was funded by ERDT-DOST in the School of Engineering at the University of San Carlos.

5.4. Acknowledgements

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5.5. Conflicts of Interest

The authors declare no conflict of interest.

6. References


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Effect of GGBFS on Workability and Strength of Alkali-activated Geopolymer Concrete

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Abstract

This paper focuses on the development of a concrete material by utilizing fly ash and blast furnace slag in conjunction with coarse and fine aggregates with an aim to reduce pollution and eliminate the use of energy extensive binding material like cement. Alternative binding materials have been tried with an aim to get rather an improved concrete material. Alkali-Activated Solution (AAS) made of the hydroxide and silicate solutions of sodium was adopted as the liquid binder whereas, Class F” fly ash and Ground Granulated Blast Furnace Slag (GGBFS) mixed in dry state were used as the Geopolymer Solid Binder (GSB). The liquid binder was used to synthesize the solid binder by thermal curing. The paper investigates the use, influence and relative quantities of the liquid and solid binders in the development of the alkali-activated GGBFS based Geopolymer Concrete (GPC). Varying ratios of AAS to GSB were taken to assess their optimum content. Further, different percentages of GGBFS were used as a partial replacement of Class F fly ash to determine the optimum replacement of GGBFS in the GPC. In order to assess their effects on various properties test samples of cubes, cylinders and beams were cast and tested at 3, 7, and 28 days. Thermal curing of GPC has also resorted for favorable results. It was found that AAS to GSB ratio of 0.5 and GGBFS content of 80% yielded the maximum strength with a little unfavorable effect on workability. The overall results indicated that AAS and GGBFS offer good geopolymer concrete which will find its applicability in water scarce areas.

Keywords: Fly Ash; GGBFS; Geopolymer Solid Binder; Alkali-Activated Solution; Geopolymer Concrete; Workability; Mechanical Properties.

1. Introduction

The most widely used material to bind the constituents of conventional concrete has been Portland cement. The gain in strength and durability properties of Portland cement concrete is also considerable. Cement production, on one hand consumes a significant amount of energy and natural raw materials and, on the other hand it liberates solid wastes and carbon dioxide (CO₂) gas which cause environmental pollution. The cement industries contribute as much as 5-7% to global CO₂ emissions [1, 2]. Massive heaps of wastes of fly ash from coal-based power plants and slag from primary units of iron industries have come up. Disposal of industrial wastes is a big challenge. The process of disposal of industrial and constructional wastes might be uneconomical but increasing demand and price of raw materials coupled with uncompensable damage to the environment have increased the importance of the utilization of these by-product wastes [3]. However, with the use of modern green engineering technologies, environment friendly...
and more energy-saving binding materials are possible. “Class F” fly ash which is a low calcium by-product with pozzolanic property [4], is formed from anthracite or bituminous coal. At an elevated temperature low calcium fly ash reacts with alkali-activated solution (AAS) which is a mixture of the hydroxide and silicate of sodium solution (NaOH & Na2SiO3). The reaction product is an inorganic aluminosilicate polymer known as geopolymer [5, 6]. Fly ash (Class F) and slag act as a geopolymer solid binder (GSB) whereas AAS acts as a geopolymer liquid binder in the GGBFS-alkali activated geopolymer concrete. Fly ash (Class F) contains more quantity of alumina and silica compounds as compared to other classes of fly ash [7]. Slag is reach in calcium oxide, silica, and alumina. The geopolymerisation process is slow at ambient temperature because of the low reactivity of the solid binder with the liquid binder [8]. The rise in the curing temperature accelerates the reactivity of the solid binder with the liquid binder. The solid binders when react with an alkali-activated solution result in sodium aluminosilicate and calcium aluminosilicate gels, respectively at thermal curing conditions. With the higher percentage of GGBFS slag content, the sodium aluminosilicate gel transforms into calcium aluminosilicate gel. The matrix of the transformed product, because of its higher density, is advantageous in improving durability and strength properties [9, 10].

In the coming years, Class F fly ash might be used as a solid binder in geopolymer to gain higher early strength and better acid and temperature resistance [7, 11]. Most of the parameters such as the concentration of sodium hydroxide solution, the ratio of silicate to hydroxide of sodium solution, the ratio of AAS to GSB, the quantity of fly ash and the curing technique affect the strength properties of the geopolymer concrete. The concentration of sodium hydroxide increases the strength of the geopolymer concrete. Many researchers [10, 12-16] have obtained the optimum ratio of the silicate to hydroxide of sodium between 1.5 to 2.5 keeping a higher molarity of sodium hydroxide (10 to 16 M) to obtain higher compressive strength. Fly ash based geopolymer concrete gains strength very slowly at an ambient temperature. However, a reasonable gain in strength has been found by resorting to oven curing in the temperature range of 40-90°C [17]. Vijai et al. (2010) and Noushini et al. (2020) [18, 19] have found that fly ash based geopolymer concrete gained maximum strength when cured in the range of 60-75°C for 24 hrs. Additives like GGBS and slag have also been used to improve the mechanical and durability properties of geopolymer concrete [1, 7, 8, 10, 20]. It has also been reported that with the use of 75% fly ash, 25% slag and 14 M concentration of NaOH in preparation of geopolymer concrete yielded a compressive strength value of 35 MPa even at 28 days of ambient curing [7]. The compressive strength of geopolymer concrete increased with the increase of slag content and concentration of NaOH solution [10, 20]. Bellum (2019) found that geopolymer concrete containing 30% fly ash and 70% GGBS yielded compressive strength of 34.15 MPa at a ratio of AAS to GSB of 0.35 and 70°C oven curing for 24Hr followed by 28 days of ambient curing [21]. Ma et al. (2019) reported a maximum compressive strength with 30% slag in geopolymer concrete. However, it also reported that the concentration of NaOH made little difference on 28 days compressive strength [22].

It is seen that most of the researchers emphasize broadly that the geopolymer concrete mixed with fly ash and alkali-activated solution (AAS) can yield the maximum compressive strength at a concentration of NaOH between 15.5 -16 M and at a ratio of silicate to hydroxide of sodium solution between 1.5-2.5. Very few investigations have reported about the mechanical properties of geopolymer concrete (GPC) containing fly ash and ground granulated blast furnace slag (GGBFS) and about the ratio of AAS to GSB.

Therefore, this research work strives to develop a concrete material composed of fly ash and blast furnace slag together with coarse and fine aggregates with an aim to reduce the environmental pollution by utilizing fly ash and blast furnace slag and also to eliminate the use of energy extensive binding material like cement. Alternative binding materials containing the hydroxide and silicate of sodium (known as the liquid binder AAS) have been tried with an aim to get an improved concrete material. Apart from the liquid binder, Class F fly ash and ground granulated blast furnace slag (GGBFS) mixed in dry state have also been used as the solid binder. This research paper focuses on the determination of the optimum quantity of GGBFS relative to fly ash to get the maximum strength and workability. Experimental investigations to determine the effect of the ratio of AAS to GSB on the compressive strength of alkali-activated GPC have been conducted. Thermal curing at 60°C for 24hours was also adopted to watch for its favourable effect. Further, the mechanical properties of GPC containing various percentages of GGBFS as a partial replacement of fly ash, in order to achieve improvement in properties, have been investigated by conducting various experiments like compressive strength, flexural strength, modulus of rigidity and split tensile strength tests.

2. Experimental Investigation

2.1. Materials

The geopolymerisation reaction between the solid binder ("Class F" fly ash and GGBFS) and liquid binder (AAS) forms the geopolymer binding material which is the counter-part of aluminosilicate in Portland cement. Tables 1 and 2 present physical properties and compositions of Class F fly ash and GGBFS. The main components of liquid binder are sodium hydroxide and sodium silicate solutions. Fine and graded coarse aggregates for use in geopolymer concrete were obtained from the local source. The physical properties and grading curve of coarse and fine aggregates (FA) of Zone III as per IS: 383 [23] are specified in Table 3 and Figure 1, respectively.

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Table 1. Physical properties of “Class F” fly ash and GGBFS

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Physical Properties</th>
<th>“Class F” fly ash</th>
<th>GGBFS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Color</td>
<td>Light brown</td>
<td>Off white</td>
</tr>
<tr>
<td>2</td>
<td>Residue retained on 45 µm, (%)</td>
<td>1.2</td>
<td>2.3</td>
</tr>
<tr>
<td>3</td>
<td>Specific surface area (Blaine), (m²/kg)</td>
<td>392</td>
<td>378</td>
</tr>
<tr>
<td>4</td>
<td>Specific gravity</td>
<td>2.23</td>
<td>2.81</td>
</tr>
<tr>
<td>5</td>
<td>Moisture content, (%)</td>
<td>0.09</td>
<td>0.11</td>
</tr>
<tr>
<td>6</td>
<td>Autoclave expansion, (%)</td>
<td>0.04</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Table 2. Chemical composition by mass % of “Class F” fly ash and GGBFS

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Chemical compounds</th>
<th>“Class F” fly Ash (%)</th>
<th>GGBFS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SiO₂</td>
<td>60.4</td>
<td>31.6</td>
</tr>
<tr>
<td>2</td>
<td>Al₂O₃</td>
<td>25.8</td>
<td>14.2</td>
</tr>
<tr>
<td>3</td>
<td>Fe₂O₃</td>
<td>4.1</td>
<td>1.7</td>
</tr>
<tr>
<td>4</td>
<td>CaO</td>
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<td>39.5</td>
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<td>5</td>
<td>MgO</td>
<td>0.8</td>
<td>5.9</td>
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<tr>
<td>6</td>
<td>SO₃</td>
<td>0.65</td>
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<td>7</td>
<td>K₂O</td>
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</tr>
<tr>
<td>8</td>
<td>Na₂O</td>
<td>0.76</td>
<td>0.5</td>
</tr>
<tr>
<td>9</td>
<td>Loss of ignition</td>
<td>2.08</td>
<td>3.7</td>
</tr>
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</table>

Table 3. Physical properties of coarse and fine aggregates

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Physical properties</th>
<th>Coarse aggregate (CA)</th>
<th>Fine aggregate (FA)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CA-I (fraction I)</td>
<td>CA-II (fraction II)</td>
</tr>
<tr>
<td>1</td>
<td>Shape</td>
<td>Angular</td>
<td>Angular</td>
</tr>
<tr>
<td>2</td>
<td>Maximum size</td>
<td>16 mm</td>
<td>12.5 mm</td>
</tr>
<tr>
<td>3</td>
<td>Water absorption</td>
<td>0.58%</td>
<td>0.71%</td>
</tr>
<tr>
<td>4</td>
<td>Surface moisture content</td>
<td>Nil</td>
<td>Nil</td>
</tr>
<tr>
<td>5</td>
<td>Specific gravity</td>
<td>2.71</td>
<td>2.69</td>
</tr>
<tr>
<td>6</td>
<td>Fineness modulus</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>7</td>
<td>Aggregate Crushing value</td>
<td>21.4%</td>
<td>22.1%</td>
</tr>
<tr>
<td>8</td>
<td>Aggregate Impact value</td>
<td>23.7%</td>
<td>24.2%</td>
</tr>
</tbody>
</table>

Figure 1. Grading curves of aggregates
2.2. Preparation of the Binder

One day before the casting of the GPC, sodium hydroxide solution was prepared. Sodium hydroxide pellets were kept in plastic vessel of tap water having 97% purity and pH value 7.12-7.20. A magnetic stirrer was used to stir thoroughly until they dissolved. Safety measures were exercised as significant quantity of heat evolved due to exothermic chemical reactions. The alkaline solution was then capped and allowed to cool. The concentration of sodium hydroxide as well as the optimum ratio of sodium silicate to sodium hydroxide was kept 16 M and 1.8 respectively, based on the results of previous studies.

The pH value and specific gravity of 16 M NaOH solution were 12.4 and 1.44, respectively. The sodium silicate solution in gel form was collected from the market. The sodium silicate solution composed of silicon dioxide (SiO$_2$)-30.4%, disodium oxide (Na$_2$O)-11.6%, water-56.9% and the remaining were filler materials. The specific gravity of sodium silicate was 1.38. The sodium silicate gel was mixed with the sodium hydroxide solution. This solution was stirred thoroughly for 5 minutes which resulted in an alkaline-activator solution (liquid binder) through an exothermic reaction [24]. This solution was kept in a tightly capped container.

2.3. Mix Proportion, Mixing, and Preparation of Sample

The mixing process of geopolymer concrete can be either through the dry mix process or wet-mix process. In this study, the dry mix process was adopted. “Class F” Fly ash, GGBFS, alkaline-activator solution (AAS) of NaOH and Na$_2$SiO$_3$ solutions, fine aggregate, coarse aggregate, and water were proportioned. To study the effectiveness of the ratio of AAS to GSB on the strength properties of fly ash based GPC, various ratios (0.40, 0.45, 0.50, and 0.55) were adopted. Further, to study the effectiveness of GGBFS in geopolymer concrete, varying proportions of GGBFS by replacing fly ash from the mix were used.

The fly ash was partially replaced by GGBFS in 20, 40, 60, 80, and 100% by the weight of fly ash in the mix. The concrete constituents were proportioned on the trial and error method because of the unavailability of the exact design procedure [15]. The mix design criterion in this study is based on the specific gravity of ingredients. The quantities and proportions of the ingredients of the GPC mix are given in Table 4. The total weight of the solid binder was kept fixed which is 460 kg/m$^3$. The absolute volume, grading curve of aggregates and specific gravity of materials have been used to determine the quantity of aggregates. The mix design methodology is presented in the form of a flowchart as shown in Figure 2.

The surface dried coarse and fine aggregates, fly ash and GGBFS were mixed in a dried state in a rotating mixer machine for 120 seconds. The AAS and water (pH=7.12-7.20) were gradually mixed together for 60 seconds and then mixed with the mixture of coarse and fine aggregates, fly ash and GGBFS continuously for further 180 seconds to achieve a uniform concrete mixture. This freshly mixed geopolymer concrete was cast in 150 mm cube moulds, 150×300 mm cylinder moulds, and 100×100×500 mm beam moulds. Compaction of concrete moulds was done on a vibration table. The concrete-filled moulds were enclosed with a plastic wrapping sheet to stop the evaporation of free water from the green concrete.

Table 4. Mix proportion of fly ash and slag based geopolymer concrete

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>W/GSB</th>
<th>AAS/GBS</th>
<th>Molarity of SH$_{sol}$</th>
<th>SS$<em>{sol}$/SH$</em>{sol}$</th>
<th>% of GGBFS by weight of GSB (Fly ash + GGBFS)</th>
<th>CA by weight of GSB</th>
<th>FA by weight of GSB</th>
</tr>
</thead>
<tbody>
<tr>
<td>M$_{0.40}$</td>
<td>0.23</td>
<td>0.40</td>
<td>16</td>
<td>1.8</td>
<td>0</td>
<td>2.47</td>
<td>1.07</td>
</tr>
<tr>
<td>M$_{0.45}$</td>
<td>0.23</td>
<td>0.45</td>
<td>16</td>
<td>1.8</td>
<td>0</td>
<td>2.44</td>
<td>1.06</td>
</tr>
<tr>
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<td>0.23</td>
<td>0.50</td>
<td>16</td>
<td>1.8</td>
<td>0</td>
<td>2.41</td>
<td>1.05</td>
</tr>
<tr>
<td>M$_{0.55}$</td>
<td>0.23</td>
<td>0.55</td>
<td>16</td>
<td>1.8</td>
<td>0</td>
<td>2.35</td>
<td>1.04</td>
</tr>
<tr>
<td>M$_{0.50}$</td>
<td>0.23</td>
<td>0.50</td>
<td>16</td>
<td>1.8</td>
<td>20</td>
<td>2.46</td>
<td>1.07</td>
</tr>
<tr>
<td>M$_{0.50}$</td>
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<td>0.50</td>
<td>16</td>
<td>1.8</td>
<td>40</td>
<td>2.51</td>
<td>1.09</td>
</tr>
<tr>
<td>M$_{0.50}$</td>
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<td>0.50</td>
<td>16</td>
<td>1.8</td>
<td>60</td>
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<td>1.11</td>
</tr>
<tr>
<td>M$_{0.50}$</td>
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<td>0.50</td>
<td>16</td>
<td>1.8</td>
<td>80</td>
<td>2.60</td>
<td>1.13</td>
</tr>
<tr>
<td>M$_{100.50}$</td>
<td>0.23</td>
<td>0.50</td>
<td>16</td>
<td>1.8</td>
<td>100</td>
<td>2.65</td>
<td>1.15</td>
</tr>
</tbody>
</table>
2.4. Slump Test

Workability of freshly mixed geopolymer concrete was determined by Slump test apparatus. The apparatus essentially consisted of a steel mould in the shape of a frustum of a cone along with a tampering steel rod. The inner diameters of the frustum at the bottom and top are 200 mm and 100 mm respectively. The height of the frustum is 300 mm. Workability was determined as per the Indian Standard (IS: 7320).

2.5. Curing of Samples

The concrete-filled moulds wrapped with plastic sheet were left at ambient temperature for 60 minutes. After 60 minutes of ambient curing, the moulds [21, 25] were kept in an oven for heat curing at a controlled temperature of 60°C for 24 hours. The oven-cured specimen moulds shown in Figure 3 were kept at an ambient temperature of 24-26°C and relative humidity of 60 ± 5% until testing.

3. Test Instruments and Experiments

3.1. Compressive Strength Test

The compressive strengths of GPC cubes were determined at 3, 7, and 28 days using a hydraulic digital compression testing machine (Figure 3.a) having a capacity of 2000 kN and the least count of 0.1 kN as per Indian Standard IS: 516 [26]. The test was conducted keeping a displacement rate of 1.4-1.6 Kg/min. Three cubes of each mix were tested and an average compressive strength value was obtained.
3.2. Flexure Test

The flexural strength test was done on a digital flexure testing machine (Figure 4.b) having a capacity of 100 kN and the least count of 0.1 kN. The flexural strength of a beam of dimensions 100×100×500 mm was determined by subjecting the beam to center point loading as per ASTM Standards ASTM C-293-02, 2002 [27]. Three beams of each mix were tested, and an average flexural strength value was obtained.

3.3. Modulus of Elasticity

An extensometer equipped with a dial gauge was mounted in the middle portion of the cylindrical specimen (Figure 4.c) to measure the deformation of the cylindrical sample [28, 29]. Cylindrical specimens were tested under uniaxial compression load at a displacement rate of 1.4-1.6 Kg/min.

3.4. Split Tensile Test

The split tensile test was done on the same compressive testing machine as per Indian standard IS:5816 [30]. Split strength was measured on 150×300 mm cylinders subjected to compression load transverse to the longitudinal axis of the cylinder (Figure 4.d). The same displacement rate of 1.4-1.6 kg/min was maintained. Three cylinders of each mix were tested at 28 days, and average values were obtained.

4. Results and Discussion

Nine different mix proportions of GGBFS and alkali based geopolymer concrete were tested. The workability and various strengths such as compressive, split, flexure strengths, as well as elastic modulus, were determined.

4.1. Effect of GGBFS and AAS/GSB on Workability

The geopolymer concrete mixes were designed with the solid binder (GGBFS and fly ash), liquid binder (Alkaline activated solution), aggregates, and water. In present research obtained quantity of water has fixed for all design mix. Obtained quantity of water has divided in two part. One part used in the preparation of AAS solution and other part used for slump. Higher AAS to GSB ratio have used more quantity of water for preparation of AAS and remaining water used for slump. This concrete mix was found to be cohesive and highly plastic for lower ratio of AAS to GSB content, because less water consumed in preparation of AAS and remaining more water used to make more workable GPC. The slump values have been obtained to optimised ratio of AAS to GSB in without GGBS design mixed as shown in Figure 5. The inclusion of GGBFS in the geopolymer concrete mix also reduced the slump values at optimized ratio of AAS to GSB. A comparative plot of the slump test result with the quantity of alkali-activated solution and inclusion of GGBFS into geopolymer concrete is shown graphically in Figure 5. The increasing percentage content of GGBFS increases the stiffness of the geopolymer concrete mix. It has been also observed that the geopolymer concrete and Portland cement concrete are rheologically different. Reactive and excess water has participated in the hydration process and slump of Portland cement concrete respectively while water is used in GPC only for preparation of AAS and gaining workability. As observed from previous studies, workability of GPC mix was decreased by adding slag [24, 31]. Superplasticizer can be added to improve workability at higher content of GGBFS in geopolymer mix. The mechanical and physical properties of the hardened concrete may be affected by workability.
Compressive Strength of GPC

In the present investigation, two broad modifications in the GPC were considered. In the first, four different alkali-activated solutions (AAS) to geopolymer solid binder (GSB) ratios were applied. These ratios were 0.40, 0.45, 0.50 and 0.55. In the second, five different replacements of fly ash by GGBFS were applied. These replacements were 20%, 40%, 60%, 80%, and 100% by weight of total fly ash. Samples of GPC with these modifications were tested. The results of compressive strengths at 3, 7, and 28 days are presented in Figures 6 and 7.

Figure 5. Slump value of geopolymer concrete of different AAS-to-GSB ratios with % GGBFS

4.2. Compressive Strength of GPC

Figure 6. Compressive strength value of geopolymer concrete with varying AAS-to-GSB ratio with % GGBFS
4.3. Impact of the Ratio of AAS-to-GSB on the Compressive Strength of GPC

A chart of compressive strengths of GPC at 3, 7, and 28 days for four AAS-to-GSB ratios is presented in Figure 6. It is seen that the maximum increase in the value of compressive strength is obtained at an AAS-to-GSB ratio of 0.5. Fly ash based GPC gives moderate strength.

4.3.1. Effect of GGBFS on Compressive Strength of GPC

To study the effect of GGBFS content in the GPC, different proportions of GGBFS were applied keeping AAS to GSB ratio fixed at 0.50. Figure 7 shows the influence of varying GGBFS on compressive strengths at 3, 7, and 28 days of curing. The compressive strength values at 28 days were found to increase by 232% and 245% respectively, over the ordinary (fly ash based) geopolymer concrete when 80% and 100% fly ash were replaced by GGBFS. Figure 7 shows that a 60% replacement of fly ash by GGBFS has increased the compressive strength moderately. The same rate of gain in strength is almost valid for 40% replacement of fly ash by GGBFS in GPC. However, a sharp rise in strength has been observed for 80% fly ash replacement by GGBFS. A very marginal rise in strength is observed at 100% GGBFS content in GPC. The optimum compressive strength was found at 80% GGBFS content in the GPC. Fly ash based GPC with zero GGBFS content has yielded a 3 days compressive strength upto to 88% of 28 days compressive strength. Addition of GGBFS influences the early strength as compared to the fly ash based GPC as shown in Figure 7. However, as compared to OPC or PPC based concrete, GGBFS-fly ash based GPC has also yielded a high early strength upto 77-86% of the 28 days strength. No surface cracks were visible after oven curing at 60°C. However, surface cracks caused by the shrinkage of the alkali-activated slag concrete have been reported by some investigators [10]. Moderate compressive strength with the participation of slag in GPC at ambient curing has been reported by some investigators [21, 25, 32-34]. Accelerated polymerization process among fly ash, GGBFS, and AAS are predominant at 60°C [25, 35]. The higher gain in compressive strength is credited to the greater content of calcium in GGBFS [35]. The GGBFS mainly contributes to the interaction of hydrates of calcium silicate, calcium aluminosilicate, and sodium aluminosilicate gels which are accountable for the increase in compressive strength.

4.4. Flexural Strength of GPC

Flexural strength represents the ability of beams to resist failure in bending. The flexural strengths of the specimens at the end of 28 days are listed in Table 5. The flexural strength was found to be influenced by the AAS-to-GSB ratio in GPC. The flexural strength value of geopolymer concrete without GGBFS was obtained maximum at an AAS-to-GSB ratio of 0.5. It is seen from Figure 8 that the optimum value of the flexural strength is obtained at 80% of GGBFS content. Normally, the compressive and flexural strengths have a strong relationship with each other. The predicted flexural strength of Portland cement concrete can be obtained by the ACI 318 Building Code [36], an expression for which can be given as:

\[
f_{rs} = 0.62 \times \sqrt[3]{f_c} \quad \text{MPa.} \tag{1}
\]
Where $f_c$ and $f_{rs}$ are the flexural and compressive strengths (MPa) respectively, of GPC at 28 days. The flexural strength of most of the specimens were found to be 2-20% more than the predicted flexural strength from Equation 2. Based on experimental results, the authors present a formula for the estimation of flexural strength of alkali-activated-GGBFS based geopolymer concrete as:

$$f_{rs} = 0.3377 \times f_c^{0.674} \text{ MPa.} \tag{2}$$

A comparison of the flexural strengths obtained by Diaz-Loya et al. (2011), ACI M318-05 (2005) and Nath & Sarker (2016) [29, 36, 37] vis. a vis. authors’ is presented in Figure 8. The expressions for estimating flexural strength are in terms of compressive strength. It is seen that the predicted value of flexural strength by Diaz-Loya et al. (2011) [29] is given as $f_{rs} = 0.69 \times \sqrt[2]{f_c}$ which gives higher values of flexural strength than the authors’ values for GGBFS content upto 60% whereas, the same gives lower values of flexural strengths for GGBFS content more than 60%. However, the expression $f_{rs} = 0.93 \times \sqrt[2]{f_c}$ obtained from [37] yields higher flexural strength than the authors’ predicted value (Equation 2). However, it is obvious that the flexural strength of alkali-activated-GGBFS based GPC is more as compared to OPC based concrete [38].

Table 5. Mechanical properties of GGBFS-alkali-activated geopolymer concrete at 28 days.

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Flexural Strength (MPa)</th>
<th>Elastic Modulus (MPa)</th>
<th>Split Tensile Strength (MPa)</th>
<th>Unit Weight (Kg/m³)</th>
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<tr>
<td>M0.40</td>
<td>2.70</td>
<td>10260.00</td>
<td>1.69</td>
<td>2375.70</td>
</tr>
<tr>
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<td>12890.00</td>
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<td>2390.81</td>
</tr>
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<td>2.97</td>
<td>13950.00</td>
<td>1.90</td>
<td>2404.44</td>
</tr>
<tr>
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<td>2.94</td>
<td>13790.00</td>
<td>2.10</td>
<td>2398.52</td>
</tr>
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<td>20200.00</td>
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<td>2411.56</td>
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<td>5.10</td>
<td>24500.00</td>
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</tr>
<tr>
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</tr>
<tr>
<td>M80.50</td>
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<td>31280.00</td>
<td>26.72</td>
<td>2539.78</td>
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<tr>
<td>M100.50</td>
<td>7.02</td>
<td>30750.00</td>
<td>26.75</td>
<td>2546.37</td>
</tr>
</tbody>
</table>

Figure 8. Comparison of measured flexural strengths of fly ash/GGBFS-alkali-activated geopolymer concrete with published research and Codal values
4.5. Modulus of Elasticity of GPC (MOE)

A comparison of the elastic modulus of “class F” fly ash and GGBFS based geopolymer concrete is presented in Figure 9. The elastic modulus values show a rising trend when the AAS-to-GSB ratio was increased up to 0.50. The modulus of elasticity values are compared with the predicted elastic moduli of Portland cement concrete by ACI Building Code [36] and “FIP Model Code” [39]. The elastic modulus of Portland cement concrete as per [39] can be estimated as:

\[ E_c = 8482.50(f_c)^{1/3} \text{ MPa.} \]  

(3)

Where, \( E_c \) is the static elastic modulus (MPa) of Portland cement concrete at 28 days. As per ACI Building Code [36], the expression for estimating static elastic modulus of Portland cement concrete (bulk density between 2375 to 2593 Kg/m\(^3\)) can be given as:

\[ E_c = 0.043 w_c^{1.5} \times f_c^{0.5} \text{ MPa.} \]  

(4)

Where \( w_c \) is the bulk density (kg/m\(^3\)). As shown in Figure 9, the authors’ experimentally obtained modulus of elasticity value is lower as compared to the modulus of elasticity estimated by Equations 3 and 4.

The experimentally determined values of elastic modulus were also compared with those obtained from Lee and Lee (2013) [10], where the expression for elastic modulus is given as \( E_c = 5300(f_c)^{2} \). Elastic modulus values as per Lee and Lee (2013) [10] were found to be higher than the authors’ experimental values for GGBFS content upto 40%, whereas, for GGBFS content more than 50% in GPC, lower than the experimental values of elastic modulus are shown (Figure 9). The elastic modulus values obtained by Lee et al. (2013), Hu et al. (2019) and Sofi et al. (2013) [10, 28, 40] were also found to be lower than the values obtained from ACI M318-05 and CEB-FIP Model Code [36, 39]. It is understood that the elastic modulus of GPC with high GGBFS content is more as compared to the GPC without GGBFS content. The reason for higher modulus of elasticity can be attributed to the increased production of the hydrates of calcium silicate and calcium aluminosilicate gels. These calcium compounds are produced in abundance as compared to the hydrate of sodium aluminosilicate gel in a highly GGBFS content GPC which causes higher elastic modulus. Some more researchers have also observed that the increased quantity of GGBFS increases the elastic modulus value of GPC [25, 28, 41].

Based on experimental results, a formula for the estimation of static elastic modulus of alkali activated-GGBFS based GPC is proposed as:

\[ E_c = 1610 \times f_c^{0.664} \text{ MPa.} \]  

(5)

Figure 9. Comparison of measured modulus of elasticity of fly ash/GGBFS based geopolymer concrete with other published research and Codal values
4.6. Split Tensile Strength Test

The split tensile strength of GPC is known to have related to some aspects of crack initiation and propagation in the concrete structure. The split tensile strength shows a rising trend when the content of GGBFS was increased keeping a constant AAS/GSB ratio of 0.50 in alkali-activated-GGBFS based geopolymer concrete. Predicted splitting tensile strength \( f_{ctm} \) of Portland cement concrete as per CEB-FIP Model Code 95 [39] and ACI 318 Building Code [36] respectively, are given by:

\[
f_{ctm} = 0.335(f_c)^{2/3} \text{ MPa.} \tag{6}
\]

\[
f_{ct} = 0.56(f_c)^{0.50} \text{ MPa.} \tag{7}
\]

where \( f_{ctm} \) and \( f_{ct} \) are the mean tensile strength value and tensile strength value of concrete at 28 days, respectively. Another study by Lee and Lee (2013) [10] showed that the splitting tensile strength value of alkali-activated-GGBFS based geopolymer concrete was 0.45 times the square root of its compressive strength. Based on experimental results (Figure 10), a formula for estimating the splitting tensile strength of alkali-activated-GGBFS based geopolymer concrete is proposed as:

\[
f_{ct} = 0.108(f_c)^{0.868} \text{ MPa.} \tag{8}
\]

As shown in Figure 10, the experimentally obtained splitting tensile strength value is lower than the value predicted by ACI M318-05 and CEB-FIP Model [36, 39] for Ordinary Portland cement concrete. But for the sample mix M0.40 and M0.50 the obtained splitting tensile value are more than the values obtained from [36]. The split tensile strength values found by Lee et al. (2013), Hu et al. (2019) and Sofi et al. (2013) [10, 28, 40]. They were also lower than the ACI M318-05 and CEB-FIP Model [36, 39] predicted values obtained from Equations 6 and 7. However, the split tensile strength values calculated according to Lee and Lee (2013) [10] were higher than the presented experimental values up to 50% GGBFS content, but those were lower than the present experimental values for more than 50% GGBFS content in GPC as shown in Figure 10.

![Figure 10. Comparison of present measured splitting tensile strength of “class F” fly ash/GGBFS geopolymer concrete with other published results and Codal values.](image)

5. Conclusions

In this study, an attempt has been made to develop alkali-activated ground granulated blast furnace slag (GGBFS) based geopolymer concrete of considerable high strength. The achievement of this strength is attributed to the alkali-activated solution (AAS) containing hydroxide and silicate solutions of sodium and GGBFS. Investigation of strength and workability revealed the following facts.
Increased quantity of GGBFS reduces the workability but increased AAS-to-GSB ratio increases the workability of the geopolymer concrete.

Increased AAS-to-GSB ratio in fly ash based GPC contributes to the increased strength. However, an increase in the ratio beyond 0.5 does not bring any appreciable change in strength. AAS-to-GSB ratio of 0.50 yields maximum compressive strength value in the fly ash based GPC.

Substantial improvement in the mechanical properties of GPC is attained with the increased content of GGBFS. It is seen that 80% GGBFS content in GPC has produced maximum strength. The rise in strength in GPC having more than 80% GGBFS content is negligible.

GPC when cured at 60° C for 24 hours attains high early strength up to 77-86% of the 28 days compressive strength.

Empirical formulae have been proposed to estimate the flexural strength, elastic modulus and split tensile strength in terms of compressive strength of GPC cured at 60°C for 24hrs. It is expected that these formulae will be helpful for the concrete technologists.

The present paper has strived to bring out a somewhat novel concrete material, based on ground granulated blast furnace slag and alkali-activated solution that possesses substantial mechanical properties while utilizing the industrial wastes. It is expected that this concrete material will find its wide applicability as structural concrete. It would be useful much more in areas where mixing water is not available.

6. Abbreviations and Nomenclature

| AAS: alkali-activated solution | CA: Coarse aggregate |
| FA: Fine Aggregate | GGBFS: Ground granulated blast furnace slag |
| GLB: Geopolymer liquid binder | GPC: Geopolymer concrete |
| GSB: Geopolymer Solid binder | OPC: Ordinary Portland cement |
| PPC: Portland Pozzolanic cement | PSC: Portland slag cement |

7. Declarations

7.1. Author Contributions

Conceptualization: G.K. and S.S.M.; methodology: G.K. and S.S.M.; validation: G.K.; formal analysis: G.K.; investigation: G.K.; resources: G.K.; data curation: G.K.; writing—original draft preparation, G.K.; writing—review and editing, S.S.M.; visualization: G.K.; supervision: S.S.M.; project administration: S.S.M.; funding acquisition: G.K. All authors have read and agreed to the published version of the manuscript.

7.2. Data Availability Statement

The data presented in this study are available in article.

7.3. Funding and Acknowledgements

The authors acknowledge the funding and support from NIT Patna and the staff members of the Cement Concrete Laboratory of the Department of Civil Engineering, NIT Patna.

7.4. Conflicts of Interest

The authors declare no conflict of interest.

8. References


[36] ACI M318-05, Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, (2005).
Marshall Performance and Volumetric Properties of Styrene-Butadiene-Styrene Modified Asphalt Mixtures

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Abstract

The durability of asphalt pavement is associated with the properties and performance of the binder. This work-study intended to understand the impact of blending Styrene-Butadiene-Styrene (SBS) to conventional asphalt concrete mixtures and calculating the Optimum Asphalt Content (OAC) for conventional mixture also; compare the performance between SBS modified with the conventional mixture. Two different kinds of asphalt penetration grades, A.C. (40-50) and A.C. (60-70), were improved with 2.5 and 3.5% SBS polymer, respectively. Marshall properties were determined in this work. Optimum Asphalt Content (OAC) was 4.93 and 5.1% by weight of mixture for A.C. (40-50) and (60-70), respectively. Marshall properties results show an increase in the stability value by 8.65 and 20.19% for A.C. (40-50) with 2.5 and 3.5% of SBS, respectively. And an increase by 9.32 and 20.61% for A.C. (60-70) with 2.5 and 3.5% of SBS respectively. Furthermore, the results indicate a decrease in Marshall flow by 14.7 and 26.47% for A.C. (40-50) with 2.5 and 3.5% SBS respectively and a decrease by 10.46 and 21.21% for A.C. (60-70) with 2.5 and 3.5% SBS respectively. Other Marshall properties were also calculated. Moreover, Blending SBS polymers to conventional asphalt mixtures produces a better performance to asphalt binder and better Marshall properties, which provides a great solution to Iraqi road problems affected by temperature and high traffic load, including less maintenance.

Keywords: Optimum Asphalt Content (OAC); Styrene-Butadiene-Styrene (SBS); Marshall Test; Volumetric Properties.

1. Introduction

Polymers and other additives are widely used to alter and enhance the asphalt cement properties, leading to an improvement asphalt mixture result. Polymers come in a variety of forms, including acrylic, elastomers, and fibres [1]. Since the 1980s, bitumen polymer modification was widely used to reduce bitumen resistance to various temperatures, allowing for drooping in typical failure techniques such as rutting and cracking [2] because of its good engineering properties and economy. SBS has been commonly used in bitumen alteration [3]. As a result, SBS can increase and expand bitumen's elasticity, making it the most suitable polymer for bitumen modification. Polymers also improve the efficiency of asphalt binders [4]. Polymer adjustment of asphalt has been a more fundamental process for treating road distresses for heavy loads, high traffic levels, and tire pressures has required user followers to examine polymer adjustment for binder asphalt application for a long time [5].

Hamdou et al. (2014) studied the influence of polymers (four types) on asphalt mixture with various asphalt content. The results show a reduction in permanent deformation, also produce development in the resilient modulus...
and more resistance to temperature [6]. High temperatures most often cause pavement deformation. The ideal binder should have consistent low-temperature susceptibility properties across the ambient temperature spectrum; thus, adding SBS polymer reduces the viscosity-temperature of bitumen in the ambit of 0-100 °C, increasing asphalt resistance to rutting [7, 8].

Isacsson and Zeng (1997), looked at the rheological properties of SBS copolymer changed bitumen and discovered that increasing the SBS percentage from 2 to 6% by asphalt weight led to that a marked improvement in the asphalt cement properties [9]. Stuart (1990) studied the effect of SBS on aggregate-asphalt interface adhesion force and discovered that adding SBS polymers increases adhesion force and increases bonding between aggregate and asphalt [10].

Pasandín et al. (2016) study properties of modified asphalt with SBS by adding seven percentages (0.5, 1.0, 1.5, 2.0, 2.5, 3.0, and 3.5) % by the asphalt weight of, and found that the SBS-modified binder mixtures improved Marshall Properties while also increasing moisture damage protection. The hot mix of modified asphalt, which contains 2.5% SBS by asphalt weight, improved Marshall’s limitations and moisture resistance [11].

Also, when compared to control mixtures, Haider A. Obaid (2015) investigated the influence of SBS on moisture damage resistance in various asphalt mixtures. He concluded that modified asphalt with an ambit (0.5-4%) of SBS has excellent properties, such as Air voids, Marshall Stability, and increasing compressive strength for unconditioned and conditioned mixtures at different content [12].

The SBS-rich phase forms, a rubbery supporting network is created in the blended bitumen, which produced the Heightened complex modulus and viscosity enhanced elastic response and development cracking resistance at low temperatures of SBS blended bitumen [13], also produced growth in rheological properties such as increase the softening point, viscosity and decrease the penetration grade [14, 15]. Figure 1 shows Fluorescent images of SBS modified asphalt with various contents of SBS [16].

![Fluorescent images of SBS modified bitumen with various contents](image)

**Figure 1.** Fluorescent images of SBS modified bitumen with various contents [16]

The conventional tests signify shows that the stiffness of bitumen increases with the addition of SBS Polymer into pure bitumen asphalt, and a significant reduction in temperature susceptibilities of bitumen asphalt take place. That indicates that SBS modified binders can especially be used in regions with high temperatures [17].

Leng et al. (2018) studied the influence of modified asphalt with SBS and concluded that adding SBS leads to a reduction in penetration from 59 to 55 mm, the softening point increased from 56 °C to 70 °C and suggested that SBS additives lead to upgrade hardness, stiffness, water-resistance, and ductility of asphalt [18].
Adding SBS to asphalt binder will produce a better asphalt mixture, including increasing asphalt viscosity, increasing film thickness and water resistance to water damage [10, 11]. Although the optimal SBS value with reasonable conclusion 3% but even those 3.5, 4% show decent results, the impact of the SBS on binder at this ambit softly alters the properties and performance of the asphalt binder mixture [19]. Polymer adjusted bituminous mixtures have been discovered to have the most significant potential for effective road construction utilization to improve the pavement's longevity and service length or reduce the road layer thickness or base thickness [20]. SBS also decreased the sensitivity to ageing and sunlight, which leads to improvement in anti-rutting performance and forming hard carbide films of surface layer [21-23].

SBS in asphalt would increase production cost, but the increased cost will be justified because the overall maintenance cost will decrease once the road construction is completed. As a result, the blend of increasing cost and reducing maintenance appears to be more cost-efficient than the lower cost at the start of the project but altitude maintenance costs. The modification of bitumen would improve performance for a more extended period on heavily trafficked roads with less maintenance access. The purpose of this research is to evaluate the impact of blending various SBS content to conventional asphalt on Marshall properties, for wearing course layer of asphalt pavement with two types asphalt grades A.C. (40-50) and A.C. (60-70), also; compare the results of modified asphalt with control mixture of asphalt pavement and discourse the results.

2. Materials and Methods

Figure 2 shows the steps of work developed in this research.

The laboratory work included asphalt binder conventional test for virgin and modified asphalt cement. Asphalt concrete specimens were made for the Marshall test to calculate the optimum asphalt and Marshall Test properties. Such as stability and flow values. Moreover, the attributes of volume were calculated to meet Iraqi requirements. The asphalt mixture that was used was formed to match the wearing course specified with the maximum size aggregate 12.5 mm. Two grades of the binder were used 40-50 and 60-70. SBS was added to asphalt with 2.5 and 3.5 percentages by asphalt weight for the two asphalt grades.

2.1. Asphalt Cement

Regarding hot mix asphalt mixtures, two grades of asphalt binder were utilized. A.C. (40-50) and (60-70) grades were provided from Al-Dura petroleum Factory. The physical features of two grades of binder can be shown in Tables 1 and 2. The results tests satisfy the specifications set out by the State Corporation for Roads and Bridges (SCRB R/9, 2003) [24].

![Flow chart of the methodology](image-url)
Table 1. Physical properties of A.C. 40-50

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<tbody>
<tr>
<td>Penetration (25°C, 100 gm, and 5 sec)</td>
<td>44 (0.1 mm)</td>
<td>40 - 50</td>
<td>D-5</td>
</tr>
<tr>
<td>Softening point, (Ring and Ball)</td>
<td>51 (°C)</td>
<td>------</td>
<td>D-36</td>
</tr>
<tr>
<td>Ductility, 25 °C and 5 cm/minute</td>
<td>152 (cm)</td>
<td>≥ 100</td>
<td>D-113</td>
</tr>
<tr>
<td>Specific Gravity @ 25 °C</td>
<td>1.048</td>
<td>------</td>
<td>D-70</td>
</tr>
<tr>
<td>Flash point, (Cleveland Open Cup)</td>
<td>294 (°C)</td>
<td>&gt;232</td>
<td>D-92</td>
</tr>
</tbody>
</table>

Table 2. Physical properties of A.C. 60-70

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (25°C, 100 gm, and 5 sec)</td>
<td>65 (0.1 mm)</td>
<td>60 - 70</td>
<td>D-5</td>
</tr>
<tr>
<td>Softening point, (Ring and Ball)</td>
<td>45 (°C)</td>
<td>------</td>
<td>D-36</td>
</tr>
<tr>
<td>Ductility, 25 °C and 5 cm/minute</td>
<td>171 (cm)</td>
<td>≥ 100</td>
<td>D-113</td>
</tr>
<tr>
<td>Specific Gravity @ 25 °C</td>
<td>1.039</td>
<td>------</td>
<td>D-70</td>
</tr>
<tr>
<td>Flash point, (Cleveland Open Cup)</td>
<td>249 (°C)</td>
<td>&gt;232</td>
<td>D-92</td>
</tr>
</tbody>
</table>

2.2. Coarse and Fine Aggregate

The coarse crushed aggregate utilized to prepare hot asphalt mixtures in this research is supplied from Al-Nibaie quarry. It retains on sieve No. 4. The fine aggregate was bought from the Ashour government Company (particle size between No.4 and No. 200). Aggregate properties were conformed to Laboratory evaluation. The physical properties of aggregates are shown in Table 3.

Table 3. The physical characteristics of Aggregates

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Designation No. [25]</th>
<th>Test Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity (Bulk)</td>
<td>C-128 &amp; C-127</td>
<td>2.63</td>
</tr>
<tr>
<td>Specific Gravity (Apparent)</td>
<td>C-128 &amp; C-127</td>
<td>2.624</td>
</tr>
<tr>
<td>Water Absorption Percent</td>
<td>C-128 &amp; C-127</td>
<td>0.961</td>
</tr>
<tr>
<td>Per cent Wear</td>
<td>C-131</td>
<td>----</td>
</tr>
</tbody>
</table>

2.3. Mineral Filler

The limestone dust used in this study to prepare the asphalt concrete mixture is passed sieve No 200, (0.075 mm). A lime factory in Karbala provides it. The physical features of the mineral filler are shown in Table 4.

Table 4. The physical properties of the limestone dust

<table>
<thead>
<tr>
<th>Property</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passing sieve No 200</td>
<td>95%</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.73</td>
</tr>
</tbody>
</table>

2.4. SBS Additive (Styrene-Butadiene-Styrene)

SBS is a thermoplastic polymer that increases asphalt pavement's overall consistency. SBS softens at high temperatures, making it simple to apply and blend with asphalt binder. SBS was brought from the Kraton Company in France and added to asphalt cement to produced asphalt mixture with good properties. Table 5 shows the physical and chemical features of SBS, and Figure 3 shows the SBS that was used in this work.

Table 5. Properties of SBS

<table>
<thead>
<tr>
<th>Property</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Physical state</td>
<td>solid</td>
</tr>
<tr>
<td>Density (Kg/m³)</td>
<td>1247</td>
</tr>
<tr>
<td>Melting point (°C)</td>
<td>197</td>
</tr>
<tr>
<td>Colour</td>
<td>yellow</td>
</tr>
</tbody>
</table>
2.5. Selection of Aggregates Gradation

The gradation aggregate in this research was selected according to the specifications recommended by the State Commission of Roads and Bridges (SCRB) [24]. Figure 4 depicts the aggregate gradation chosen due to the wearing course, with an aggregate nominal maximum size of 12.5 mm.

![Figure 3. SBS used in experimental models](image)

![Figure 4. Aggregates gradation for the wearing course](image)

3. Marshall Test

This test intended to find the (OAC), stability, flow, density, and air voids for asphalt concrete mixture by cylindrical shape specimens that exhibited 4 inches in diameter and 2.5 inches in height. The bulk specific gravity and density were determined in compliance with ASTM-D2726. The per cent of air voids were calculated for each specimen according to ASTM D3203, the per cent of air voids were calculated from Equation 1:

\[
\% \text{ Air Voids} = \left[1 - \frac{\text{Bulk sp. gr.}}{\text{Max. Theo. sp.gr.}}\right] \times 100
\]

(1)

Where:
- Bulk sp. gr. ⇒ Bulk Specific Gravity.
- Max. Theo. sp. gr. ⇒ Maximum Theoretical Specific Gravity of the Mixture.

For each specimen, Marshall Stability and flow values were determined using the procedure outlined in ASTM D6927.

3.1. Preparation of Conventional and Modified Mixture

To obtain asphalt concrete mixture according to wearing course specifications. The aggregate was sieved and separated for each size, then mix with the filler to obtain suitable gradation meeting with specifications. This combined filler and aggregate were the temperature raised to 155 °C, while asphalt was being heated at the same time to a temperature that results in a kinematic viscosity of (170±20) centistokes. The aggregate with a filler and different amounts of virgin asphalt binder in the mix for about two minutes ensures that asphalt binder coated aggregates surface.
For blended asphalt binder, the SBS was added to the virgin binder with 2.5 and 3.5% by asphalt weight due to conclusions of [11, 12], which study and calculate that the optimum content of SBS that blended with asphalt binder. SBS was powdered in a strong mixer before mixing with virgin asphalt, then mixed with asphalt cement at a temperature of 170 °C for 30 min. and stirring (on a hot plate) to achieve a homogenous blend. Figure 5 and 6 shows a group of Marshall specimens and Marshall test respectively.

After that, specimens in a template placed to phlegmatic at the natural temperature of the room for one day and then removed from a template using an extractor. And put in a water bath at 60 °C for a half-hour, then pressed on the side surface at a fixed rate of (50.8. mm/min) 2 in/min, until reached the failure load.

![Figure 5. Group of Marshall Specimens](image)

![Figure 6. Marshall test](image)

### 4. Results and Discussions

#### 4.1. Marshall Test

To estimate optimum asphalt content (OAC) for asphalt mixtures. Many results of Marshalls test should be obtained (Marshall stability, density-voids analysis, and flow) with using aggregate (12.5 mm nominal maximum size gradation) for wearing course, limestone dust (seven percent by weight of the total aggregate), and five various asphalt contents for each (40-50) and (60-70) penetration grade ranging from 4 to 6 percent (by a total of mix weight) with a raise of 0.5 percent. For modified asphalt, two percent of SBS (2.5 and 3.5) by weight of asphalt have been added to the virgin asphalt.

The OAC was 4.93% for virgin A.C. (40-50), 4.86% for modified A.C. (40-50) with 2.5% SBS, (4.8) % for modified A.C. (40-50) with 3.5% SBS. The OAC for conventional A.C. 60-70 was 5.1%, 4.95% for modified A.C. 60-70 with 2.5% SBS, 4.88% for modified A.C. 60-70 with 3.5% SBS, adding SBS to asphalt lead to increase in asphalt viscosity that causing reduction in aggregate absorption and increasing in free asphalt in the mixture [11]. The Marshall assets for OAC met all of the checks and met Iraq’s requirements (SCRB, 2003) [24].

The results of Marshall stability show that the stability was increased by 8.65 and 20.19% in comparison to a traditional mixture A.C. (40-50) when it applies to modified asphalt A.C. (40-50) with 2.5 and 3.5% SBS, respectively. The stability was increased by 8.9 and 19% compared to a traditional mixture A.C. (60-70) when it applies to modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS. This increase in stability can be explained as adding more SBS to asphalt binder leads to development in the performance of thermo-rheological state and stiffness related to increase of asphalt viscosity [11, 12]. The effect of SBS on Marshall stability is depicted in Figure 7.

For Marshall flow, the results show that the Marshall Flow was dropped by 14.7 and 26.47% in comparison to a traditional mixture A.C. (40-50) when it applies to modified asphalt A.C. (40-50) with 2.5 and 3.5% SBS respectively. The flow was decreased by 10.46 and 21.21% compared to a traditional mixture A.C. (60-70) when it applies to modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS. This decrease in flow can be explained as SBS's improved adhesion and cohesion between asphalt and aggregate, causing increment in asphalt viscosity [11, 20]. Figure 8 shows the impact of SBS on the Marshall flow.

For bulk density, the results show that the bulk density was increased by 0.6 and 0.86% in comparison to a traditional mixture A.C. (40-50) when it applies to modified asphalt A.C. (40-50) with 2.5 and 3.5% SBS, respectively. The bulk density increased by 0.39 and 0.91% compared to a traditional mixture A.C. (60-70) when
applies to modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS, respectively. Adding SBS to asphalt binder led to a rise in the viscosity of modified asphalt, which is responsible for decreasing aggregate absorption, causing drops in aggregate demand, leading to a reduction in bulk density [12], as shown in Figure 9.

For Air Voids, the results show that A.V. % was decreased by 2.93 and 4.8% compared to a traditional mixture A.C. (40-50) when it applies to modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS, respectively. The A.V. % was decreased by 5.5, 9.64 % compared to a traditional mixture A.C. (60-70) when it applies to modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS respectively, as shown in Figure 10.

The results show the VFA % was decreased by 1.33 and 3.33% compared to a traditional mixture A.C. (40-50) when it applies to modified asphalt A.C. (40-50) with 2.5 and 3.5% SBS respectively. The VFA % was decreased by 3.84 and 6.02 % compared to a traditional mixture A.C. (60-70) when it applies to modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS respectively. That results can be shown in Figure 11.

The results show the VMA % was increased by 2.61 and 3.62% compared to a traditional mixture A.C. (40-50) when it applies to modified asphalt A.C. (40-50) with 2.5 and 3.5% SBS, respectively. The VMA was increased by 3.8 and 4.33% compared to a traditional mixture A.C. (60-70) when it applies to modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS respectively, as shown in Figure 12.
Where A represents A.C. (40-50), B represents modified A.C. (40-50) with 2.5% SBS, C represents modified A.C. (40-50) with 3.5% SBS, D represents to A.C. (60-70), E represents to modified A.C. (60-70) with 2.5% SBS and F represents to modified A.C. (60-70) with 3.5% SBS. Finally, the effect of blending SBS polymer to asphalt leads to an increase in the thermo-rheological performance of asphalt, which leads to an increase in asphalt viscosity that is responsible for dropping aggregate absorption to the binder, causing increment in free asphalt in the total mixture and more voids in mineral aggregates [26].

5. Conclusions

This research study the performance of blend asphalt with various contents of SBS polymer as Compliant with the test program, the points below are concluded:

- The (OAC) for modified asphalt A.C. (40-50) with 2.5 and 3.5% SBS decreased by 1.42 and 2.63% compared with (OAC) of conventional asphalt A.C. (40-50), respectively. The OAC for modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS decreased by 2.94 and 4.31% compared with (OAC) of conventional asphalt A.C. (60-70), respectively.

- The results of Marshall Stability increased by 8.65 and 20.19% of modified asphalt A.C. (40-50) with 2.5 and 3.5% SBS compared with conventional mixtures A.C. (40-50) respectively. The results of Marshall Stability were improved by 9.32 and 20.61% of modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS compared with conventional mixtures A.C. (60-70), respectively.

- The results of Marshall Flow decreased by 14.7 and 26.47% of modified asphalt A.C. (40-50) with 2.5 and 3.5% SBS compared with conventional mixtures A.C. (40-50), respectively. The results of Marshall Flow improved by 10.46 and 21.21% of modified asphalt A.C. (60-70) with 2.5 and 3.5% SBS compared with conventional mixtures A.C. (60-70), respectively.

- The bulk density is raised when asphalt content increases until it reaches the peak, then begins to Famish. The additional bitumen forming more films surround the aggregates and a propensity to drive the aggregate further apart, therefore producing lower density. The bulk specific gravity increases when the aggregate demand for asphalt decreases due to increasing viscosity, leading to less absorption to the asphalt binder.

- The drop in A.V.% was found because the addition of SBS increased the solid mass fraction and the rigidity of asphalt and produced a modified binder with more brittle and viscous. When asphalt viscosity increased, absorption of aggregate to asphalt decreased and produced asphalt mixture with thicker asphalt coating aggregates. This operation was responsible for the variation of A.V.% in the asphalt mixture.

- The decrease in voids filled with asphalt percent occurred because of increased asphalt binder viscosity due to blending more SBS polymers, which finally decreased aggregate absorption to asphalt binder.

- Modified asphalt produced better Marshall properties. The increase in SBS polymer leads to improved Marshall’s properties, A.C. (40-50) for conventional asphalt and modified asphalt showing more sensitivity to SBS polymer and more improvement to Marshall properties.

Blending SBS polymers with asphalt leads to development in asphalt properties that produce great solutions for pavement problems such as (rutting, moisture damage, temperature, …) also produced development in rheological
properties (softening point and viscosity). This study recommended using SBS to develop the performance of pavement roads in Iraq.

6. Declarations

6.1. Author Contributions

Conceptualization, M.Q.I.; methodology, M.Q.I.; software, S.A.J.; writing—original draft preparation, S.A.J.; writing—review and editing, M.Q.I.; supervision, M.Q.I.; All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

The data presented in this study are available in article.

6.3. Funding

The Ministry of Higher Education and Scientific Research and Baghdad University, College of Engineering’s, Civil Engineering Department, sponsored this study.

6.4. Conflicts of Interest

The authors declare no conflict of interest.

7. References


A Macroscopic Traffic Model Based on the Safe Velocity at Transitions

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Abstract

The increasing volume of vehicles on the road has had a significant impact on traffic flow. Congestion in urban areas is now a major concern. To mitigate congestion, an accurate model is required which is based on realistic traffic dynamics. A new traffic model is proposed based on the conservation law of vehicles which considers traffic dynamics at transitions. Traffic alignment to forward conditions is affected by the time and distance between vehicles. Thus, the well-known Lighthill, Whitham, and Richards (LWR) model is modified to account for traffic behavior during alignment. A model for inhomogeneous traffic flow during transitions is proposed which can be used to characterize traffic evolution. The performance of the proposed model is compared with the LWR model using the Greenshields and Underwood target velocity distributions. These models are evaluated using the Godunov technique and numerical stability is guaranteed by considering the Courant, Friedrich, and Lewy (CFL) condition. The results obtained show that the proposed model characterizes the flow more realistically, and thus can provide better insight into traffic behavior for use in controlling congestion and pollution levels, and improving public safety.

Keywords: Macroscopic Traffic Flow; Inhomogeneous Flow; Reaction Distance and Time; Safe Velocity.

1. Introduction

The economic growth of a country is affected by the road infrastructure. Congestion wastes significant time and degrades this growth. The pollution due to the corresponding emissions causes serious health problems and reduces the quality of life. Smart traffic infrastructure is required to mitigate this congestion and address the corresponding issues. Effective traffic management requires realistic traffic characterization to predict infrastructure use [1]. This paper considers the time and distance between vehicles for alignment to forward conditions. When a stimulus is perceived, a driver reacts during the reaction time and then aligns to forward vehicles during the transition time. The reaction distance is covered during the reaction time, whereas the transition distance is covered during the transition time. Safe time includes the transition and reaction times required for safe alignment. This can be considered the minimum time needed to avoid accidents. The safe distance is covered during the safe time and includes the reaction and transition distances. The equilibrium velocity distribution corresponds to a homogeneous traffic flow with no transitions. This distribution depends on the vehicle density as well as driver behavior and road conditions [2].

The safe distance and time as well as the maximum density and velocity affect the transition behavior of traffic. Thus, a traffic flow model based on these parameters can be used to investigate traffic behavior and forecast traffic
conditions, thus helping to mitigate congestion and reduce pollution. For example, real-time information can be stored in roadside units for communication to nearby vehicles to warn of congestion ahead and lower the potential for accidents. Suggestions can be provided to drivers to adjust their speed and/or take alternate routes.

Velocity \( v \) and density \( \rho \) determine the macroscopic spatial and temporal traffic evolution. Density is the number of vehicles per unit length and traffic flow \( q(\rho) \) is the product of density and velocity. Lighthill and Whitham (1955), and Richards (1956) [3, 4] proposed the LWR model which assumes changes in flow are small and traffic alignment is instantaneous [5]. This model ignores inhomogeneous traffic flow behavior. However, an inhomogeneous traffic flow can occur whenever traffic conditions vary between locations on a road [6], and this will affect the velocity and density.

The Payne (1971) model considers vehicle conservation when there is no acceleration and includes driver presumption and relaxation when acceleration does occur. Presumption is driver anticipation to a stimulus ahead while relaxation is traffic alignment to this stimulus [7]. Whitham (2011) independently characterized traffic based on similar assumptions [8], so it is called the Payne-Whitham (PW) model. This model assumes large changes in flow do not occur and traffic variations are smooth [9]. Due to this inadequate characterization, velocity and density evolution can be unrealistic when the changes are large [5, 10]. This can occur in situations such as the sudden application of brakes in anticipation of an accident.

Del Castillo et al. (1994) improved the PW model by considering driver presumption and reaction time for small transitions in velocity and density [2]. A concavity condition was employed which requires a decrease in flow for a large density and vice versa. Further, the traffic flow versus density is assumed to be concave both spatially and temporally, and a decreasing density is needed to make the behavior realistic. Daganzo (1955) showed that the PW model can have negative velocities at abrupt changes in density. This is because traffic will occupy empty spaces which means movement opposite to the direction of flow, resulting in the velocity being negative (which is fluid-like behavior). It has been argued that traffic is anisotropic, i.e. forward traffic conditions contribute to changes in flow and this flow is not influenced by rearward vehicles [9]. Thus, an anisotropic model was developed based on driver interaction which considers both forward and rearward traffic conditions. To smooth variations in the traffic flow, the LWR model employs diffusion (viscosity) terms based on the velocity and density which are \( \frac{\partial v}{\partial x} \) and \( \frac{\partial \rho}{\partial x} \), respectively. However, these terms can create negative velocities at discontinuities. The Del Castillo et al. model may also produce unrealistic results at discontinuities [11]. This is because the anticipation and reaction time in this model are too large.

Markos (1998) [13] countered the arguments in Del Castillo et al. (1994) and Daganzo (1955) [2, 9] and suggested that the average velocity can be employed for vehicles in a macroscopic flow. Differences in velocity are due to microscopic inhomogeneous traffic conditions so changes in velocity can occur anywhere in the flow. Further, macroscopic traffic models are a simplification of microscopic traffic flow which is affected by vehicle sizes, distances between vehicles, and driver behavior, and thus these factors should be considered. In addition, negative flow can be avoided by allowing only positive velocities.

Aw and Rascle (2000) characterized traffic evolution based on the arguments of Daganzo (1955) to overcome the deficiencies of the PW model [14]. Their model consists of two coupled equations. The first characterizes conservation of vehicles and the second determines acceleration. With this model, driver presumption is a monotonically increasing function of density. However, changes in velocity are ignored so the acceleration can be high for a large density. Berg, Mason, and Woods (BMW) [15] proposed a car-following model based on headway. Headway is the distance between vehicles required for alignment and is given by \( \chi = \frac{1}{\rho} \). It is small when the density is large which results in greater interactions between vehicles. The BMW model employs a diffusion (viscosity) term based on acceleration, but it ignores the time and distance required for alignment.

Traffic models should consider driver physiology [16]. Further, the velocity during alignment to forward conditions should be included Khan et al. (2020) [17]. Driver reaction has been considered to provide more realistic behavior than existing models [18, 19]. Further, realistic parameters were used to better characterize traffic [20]. Thus, the LWR model is improved in this paper by including the transition behavior of traffic. The safe distance and safe time are considered based on the anticipated velocity, and the flow is slow with a large safe distance. Further, the traffic density distribution differs according to the safe distance and has a larger variance when this distance is smaller. The change in this distribution during a transition depends on the velocity changes required to achieve a homogeneous flow and maintain the safe distance. Transitions occur because of traffic bottlenecks, ramps, and traffic control lights, and result in an inhomogeneous traffic flow. Conversely, if a transition does not occur, the flow should be homogeneous.

The rest of this paper is organized as follows. Section 2 presents the LWR and proposed traffic models. The Godunov scheme is employed in Section 3 for numerical evaluation of these models. A comparison of the LWR and proposed models is presented in Section 4. Finally, some concluding remarks are given in Section 5.
2. Traffic Flow Models

Traffic flow dynamics should be considered in developing traffic flow models. Figure 1 shows the steps in this development. First, a model framework is defined based on qualitative statements and traffic observations. Second, the related traffic characterization literature and physical laws are used to obtain a model. Then, the performance of the model is evaluated numerically. These results are used to modify the model as required to obtain an acceptable traffic characterization [18]. The LWR and proposed traffic models are presented below.

![Figure 1. The steps in the development of a traffic flow model](image)

The LWR model is based on the principle of conservation of matter and is given by Lighthill and Witham (1995), and Richards (1956) [3, 4]:

\[
(\rho)_t + (\rho v(\rho))_x = 0,
\]

where \(\rho\) is density and \(v(\rho)\) is the equilibrium velocity distribution. The subscript \(t\) denotes partial derivative with respect to time and \(x\) denotes partial derivative with respect to space. This model assumes vehicle conservation on a road so there are no exits or entrances. A smooth traffic density is also presumed. Traffic following the equilibrium velocity distribution results in a homogeneous flow. This distribution is determined by the density distribution which characterizes traffic behaviour on a very long (infinite length), ideal road [21]. An ideal road does not have any disturbances to the traffic flow. The LWR model assumes vehicles adjust their velocity in zero time which is unrealistic. As a consequence, this model cannot be used to evaluate traffic behaviour during transitions. Further, velocity adjustments made during a transition result in an inhomogeneous flow, which is not possible with the LWR model [5].

A new traffic model is proposed which incorporates traffic behaviour during transitions. Traffic adjusts to the equilibrium velocity distribution according to the anticipated change in velocity [27]. This change can be characterized as an acceleration [16] given by:

\[
a(\rho) = \frac{v(\rho)^2 - v_a^2}{2d_s},
\]

where \(v_a\) is the average velocity at the transition, \(d_s\) is the safe distance and \(t_s\) is the safe time. \(v(\rho)\) is the velocity distribution that traffic adjusts to when a transition occurs. For the LWR model, this distribution [22] can be expressed as:

\[
v(\rho) = a(\rho) t_s.
\]

and substituting this in (1) gives:

\[
(\rho)_t + (a(\rho) t_s)_x = 0.
\]

Now substituting (2) in (4) results in:

\[
(\rho)_t + (\rho \left( \frac{v(\rho)^2 - v_a^2}{2d_s} \right) t_s) = 0.
\]

Several equilibrium velocity distributions have been proposed in the literature [23]. The Greenshields distribution [24] is widely used and is given by:

\[
v(\rho) = v_m \left( 1 - \frac{\rho}{\rho_m} \right),
\]
where $\rho_m$ is the maximum density and $v_m$ is maximum velocity. This indicates that density and velocity are inversely related. The Underwood distribution [25] is also commonly employed and can be expressed as:

$$u(\rho) = v_m \exp \left( \frac{\rho_m}{\rho} \right).$$

(7)

This is an exponential velocity distribution based on density. Substituting (6) into (5) gives:

$$\rho_t + \left( \rho \left( \left( v_m \left( 1 - \frac{\rho}{\rho_m} \right) \right)^2 - v_a^2 \right) \frac{t_s}{2d_s} \right)_x = 0.$$  

(8)

The safe velocity is $v_s = \frac{d_s}{t_s}$ so (8) can be written as:

$$\rho_t + \left( \left( v_m \left( 1 - \frac{\rho}{\rho_m} \right) \right)^2 - v_a^2 \right) \frac{\rho}{2v_s} = 0,$$

(9)

so the traffic flow during a transition is:

$$q(\rho) = \left( v_m \left( 1 - \frac{\rho}{\rho_m} \right)^2 - v_a^2 \right) \frac{\rho}{2v_s}.$$  

(10)

This indicates that vehicles maintaining a large safe distance will have slow transitions and few interactions between vehicles, whereas a small safe distance will result in fast transitions and many interactions between vehicles. If there is no transition, $v_a = 0$ can be assumed so the traffic flow becomes:

$$q(\rho) = \left( v_m \left( 1 - \frac{\rho}{\rho_m} \right)^2 \right) \frac{\rho}{2v_s}.$$  

(11)

Thus, the flow with the equilibrium velocity distribution depends on the safe velocity which is not accounted for in the LWR model. If the safe velocity is reduced by a factor $\beta$, then $v_s(\rho) = \frac{v(\rho)}{\beta}$, which gives

$$q(\rho) = \frac{\beta}{2} \left( v_m \left( 1 - \frac{\rho}{\rho_m} \right) \right) \rho,$$

(12)

and using (6) results in:

$$q(\rho) = \frac{\beta}{2} \rho u(\rho).$$  

(13)

Then from (13) and (9) we have:

$$q(\rho) + \frac{\beta}{2} \left( \rho u(\rho) \right)_x = 0,$$

(14)

which shows that the traffic flow increases as the safe velocity is decreased. The traffic flow reduces to the LWR model flow with $\beta = 2$ as substituting $v_s(\rho) = \frac{v(\rho)}{2}$ in (11) gives:

$$q(\rho) = \left( v_m \left( 1 - \frac{\rho}{\rho_m} \right) \right) \rho,$$

(15)

and using (6) results in:

$$q(\rho) = \rho u(\rho).$$  

(16)

Then combining (15) and (16) with (9) gives the LWR model:

$$\rho_t + \left( \rho u(\rho) \right)_x = 0.$$  

(17)

Unlike the LWR model, the proposed model can account for both homogeneous and inhomogeneous traffic flows.
3. Performance Evaluation

Consider a road divided into \( N \) equidistant segments and \( M \) equal duration time steps. The total length is \( x_N \) so a segment has length \( h = x_N / N \), and the total time duration is \( t_M \) so a time step is \( k = t_M / M \). The average traffic density \( \rho \) and flow \( q(\rho) \) are evaluated for the \( n \)th road segment denoted \( x_{n-h} \) to \( x_{n+h} \) over time \( t_m \) to \( t_{m+1} \) using the technique developed by Godunov [26]. The number of vehicles present in the \( n \)th segment at time \( t \) is given by:

\[
l_n(t) = \int_{x_{n-h}}^{x_{n+h}} \rho(x,t) \, dx,
\]

so the traffic flow in this segment at time \( t \) is:

\[
\Delta l_n(t) = q \left( \rho \left( x_{n-h}, t \right) \right) - q \left( \rho \left( x_{n+h}, t \right) \right) .
\]

The traffic flow in the \( n \)th segment during the time interval \((t_m, t_{m+1})\) is then:

\[
l_n(t_m + 1) - l_n(t_m) = \int_{t_m}^{t_{m+1}} \Delta l_n(t) \, dt = \int_{t_m}^{t_{m+1}} q \left( \rho \left( x_{n-h}, t \right) \right) - q \left( \rho \left( x_{n+h}, t \right) \right) \, dt,
\]

and using (18), this has the form:

\[
\int_{x_{n-h}}^{x_{n+h}} \rho(x, t_m + 1) \, dx - \int_{x_{n-h}}^{x_{n+h}} \rho(x, t_m) \, dx = \int_{t_m}^{t_{m+1}} q \left( \rho \left( x_{n-h}, t \right) \right) - q \left( \rho \left( x_{n+h}, t \right) \right) \, dt.
\]

The average density at time step \( m \) for the \( n \)th segment is:

\[
\rho(n, m) = \frac{1}{h} \int_{x_{n-h}}^{x_{n+h}} \rho(x, t_m) \, dx,
\]

and the corresponding flow is:

\[
\rho(n, m) = \frac{1}{h} \int_{t_m}^{t_{m+1}} q \left( \rho \left( x_{n-h}, t \right) \right) \, dt.
\]

Substituting (22) and (23) into (21) gives:

\[
\rho(n, m + 1) - \rho(n, m) = \frac{k}{h} \left( q(n, m) - q(n + 1, m) \right).
\]

For the LWR model, \( q(\rho) = \rho v(\rho) \), and for the proposed model \( q(\rho) \) is given by (9). The traffic flow has initial density distribution \( \rho_0(x) \) at \( t = 0 \), and this is used to determine the initial average densities. For the time interval \((t_m, t_{m+1})\) set:

\[
\rho(x, t) = \rho(n, m) \quad \text{for} \quad x_{n-h} < x < x_{n+h}.
\]

To account for both increasing and decreasing flows, \( q(\rho(x, t)) \) is approximated as:

\[
q(\rho(x, t)) = \begin{cases} 
q \left( \min(p(n-1,m), \rho(n, m)) \right), & \text{if} \quad \rho(n-1,m) \leq \rho(n, m) \\
q \left( \max(p(n,m), \rho(n-1,m)) \right), & \text{if} \quad \rho(n-1,m) > \rho(n, m).
\end{cases}
\]

For numerical stability, the Courant-Friedrichs-Lewy (CFL) condition is applied so that the maximum distance traffic covers during a time step is not greater than \( h \) so that

\[
|q'(\rho)|_{\text{max}} \times k < h.
\]

where \( |q'(\rho)|_{\text{max}} \) is the maximum rate of change at \( t = 0 \) given by:

\[
\max \left( \frac{q(\Delta \rho)}{\Delta \rho} \right) = \max \left( \frac{q(\rho(n,0)) - q(\rho(n-1,0))}{\Delta \rho} \right).
\]
\[ k = 0.5 \times \frac{h}{\left| q'(\rho) \right|_{\text{max}}}. \quad (29) \]

### 4. Simulation Results

The simulation parameters are summarized in Table I. Traffic is observed over a period of three seconds while traversing a road from −20 m to 200 m. The road begins at −20 m so that the traffic can begin uniformly distributed about 0. The length of road considered is \( x_M = 220 \text{ m} \) with \( M = 450 \) so that \( h = 0.489 \text{ m} \). The maximum velocity is \( v_m = 30 \text{ m/s} \) and the maximum normalized density is 0.2, i.e. 20% of the road is occupied. The initial traffic density distribution is \( \rho_0(x) = 0.09 \exp\left(\frac{-x^2}{50}\right) \) so that the density is uniformly distributed around 0. This distribution is used for both the LWR and proposed models. The initial density interval is set to \( \Delta \rho = 0.0004 \) to evaluate \( \left| q'(\rho) \right|_{\text{max}} \), and this is used in (29) to determine \( k \). Non-periodic boundary conditions are employed so that vehicles can move beyond the 200 m point. The traffic density evolution on the road over time is determined using the Godunov technique presented in Section 3. Two average transition velocities, \( v_a = 0 \text{ m/s} \) and 10 m/s, are considered with the equilibrium velocity distributions (6) and (7) and \( v_m = 30 \text{ m/s} \).

<table>
<thead>
<tr>
<th>Name</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average transition velocity</td>
<td>( v_a )</td>
<td>0, 10 m/s</td>
</tr>
<tr>
<td>Equilibrium velocity distribution</td>
<td>( v(\rho) )</td>
<td>Greenshields and Underwood</td>
</tr>
<tr>
<td>Maximum velocity</td>
<td>( v_m )</td>
<td>30 m/s</td>
</tr>
<tr>
<td>Initial density distribution</td>
<td>( \rho_0(x) )</td>
<td>0.09 ( \exp\left(\frac{-x^2}{50}\right) )</td>
</tr>
<tr>
<td>Length of road</td>
<td>( X )</td>
<td>220 m</td>
</tr>
<tr>
<td>Number of road steps</td>
<td>( M )</td>
<td>450</td>
</tr>
<tr>
<td>Segment length</td>
<td>( h )</td>
<td>( 220/450 = 0.489 ) m</td>
</tr>
<tr>
<td>Safe velocity</td>
<td>( v_s )</td>
<td>10, 20 m/s</td>
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<tr>
<td>Maximum normalized density</td>
<td>( \rho_m )</td>
<td>0.2</td>
</tr>
<tr>
<td>Initial density interval</td>
<td>( \Delta \rho )</td>
<td>0.0004</td>
</tr>
<tr>
<td>LWR model time step with the Greenshields distribution</td>
<td>( k )</td>
<td>0.0067 s</td>
</tr>
<tr>
<td>LWR model time step with the Underwood distribution</td>
<td>( k )</td>
<td>0.0015 s</td>
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<tr>
<td>Proposed model time step with the Greenshields distribution, ( v_a = 0 \text{ m/s}, v_s = 20 \text{ m/s} )</td>
<td>( K )</td>
<td>0.0102 s</td>
</tr>
<tr>
<td>Proposed model time step with the Greenshields distribution, ( v_a = 0 \text{ m/s}, v_s = 20 \text{ m/s} )</td>
<td>( k )</td>
<td>0.0091 s</td>
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<td>Proposed model time step with the Underwood distribution, ( v_a = 0 \text{ m/s}, v_s = 20 \text{ m/s} )</td>
<td>( K )</td>
<td>0.000256 s</td>
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<tr>
<td>Proposed model time step with the Underwood distribution, ( v_a = 0 \text{ m/s}, v_s = 20 \text{ m/s} )</td>
<td>( k )</td>
<td>0.000254 s</td>
</tr>
<tr>
<td>Total simulation time</td>
<td>( t_M )</td>
<td>3 s</td>
</tr>
</tbody>
</table>

Figure 2 shows the traffic density evolution with the LWR model at 0 s, 1.5 s, and 3 s. The target to align to forward conditions is the Greenshields distribution. The initial density is shown in blue. At 1.5 s, traffic spans from 18 m to 64 m while the maximum density is 0.060. At 3 s, the traffic spans from 50 m to 110 m and the maximum density is 0.042.
Figures 3 and 4 show the proposed model traffic density evolution for safe velocities $v_a = 20$ m/s and 10 m/s, respectively, with $d_s = 10$ m. The safe distances for these velocities are then 20 m and 10 m, respectively. With this model, traffic adjusts from $v_a = 10$ m/s to the Greenshields distribution which is the target equilibrium velocity distribution. The results in these figures show that traffic moves slower with a 20 m/s safe velocity compared to a 10 m/s safe velocity. At 1.5 s, the traffic in Figure 3 spans from 23 m to 83 m, whereas in Figure 4 it spans from 65 m to 155 m. Thus, the traffic density has a greater variance at a lower safe velocity, so this velocity has a significant effect on traffic behaviour. This variance is greater than that with the LWR model shown in Figure 2. The average distance covered is higher at a lower safe velocity as vehicles maintain a smaller safe distance. Figure 5 shows the proposed model density evolution from $v_a = 0$ m/s to the equilibrium velocity distribution with a safe velocity of 20 m/s. There are no significant differences between Figures 3 and 5, which indicates that the average transition velocity has little effect on traffic behaviour.

Figure 3. Traffic density evolution with the proposed model for $d_s = 20$ m, $v_s = 20$ m/s, $v_a = 10$ m/s, and $t_s = 1$ s. The target equilibrium velocity distribution is the Greenshields distribution.

Figure 4. Traffic density evolution with the proposed model for $d_s = 10$ m, $v_s = 10$ m/s, $v_a = 10$ m/s, and $t_s = 1$ s. The target equilibrium velocity distribution is the Greenshields distribution.

Figure 5. Traffic density evolution with the proposed model for $d_s = 20$ m, $v_s = 20$ m/s, $v_a = 0$ m/s, and $t_s = 1$ s. The target equilibrium velocity distribution is the Greenshields distribution.
Figure 6. Traffic density evolution with the proposed model for $d_s = 20$ m, $v_s = 20$ m/s, $v_a = 0$ m/s, and $t_s = 1$ s. The target equilibrium velocity distribution is the Underwood distribution.

Figure 7. Traffic density evolution with the proposed model for $d_s = 20$ m, $v_s = 20$ m/s, $v_a = 10$ m/s, and $t_s = 1$ s. The target equilibrium velocity distribution is the Underwood distribution.

Figures 6 and 7 show the proposed model traffic density evolution for $v_a = 0$ m/s and 10 m/s, respectively, at 0 s, 1.5 s and 3 s. The target equilibrium velocity distribution is the Underwood distribution and the safe time is 1 s. With $v_a = 0$ m/s, no transition occurs, but with $v_a = 10$ m/s, traffic aligns to the forward conditions. At 1.5 s, the traffic in Figure 6 spans from 50 m to 120 m, whereas in Figure 7 it spans from 40 m to 110 m. At 3 s, the traffic in Figure 6 spans from 110 m to beyond 200 m, whereas in Figure 7 it spans from 100 m to 190 m. Thus, the density variance is larger with no transition than with $v_a = 10$ m/s. Figures 3 to 7 show that the traffic evolution with the proposed model and the Greenshields or Underwood target equilibrium velocity distributions is smooth.

Figure 8. Traffic density evolution with the LWR model when the target equilibrium velocity distribution is the Underwood distribution.
Figure 8 shows the traffic density evolution with the LWR model at 0 s, 1.5 s, and 3 s. The target equilibrium velocity distribution is the Underwood distribution. At 1.5 s, the traffic spans from 30 m to 75 m whereas with the Greenshields distribution it spans from 18 m to 64 m as shown in Figure 2. At 3 s, the traffic span with the Underwood distribution is from 70 m to 130 m, whereas with the Greenshields distribution the span is from 50 m to 110 m. Thus, the density variance is greater with the Underwood distribution.

The traffic behaviour of the LWR and proposed models varies greatly as shown in Figures 2 to 8. Figures 3 to 7 show that with the proposed model, the density varies according to the target velocity distribution and safe velocity at transitions. The density with the LWR model only varies according to the target velocity distribution and ignores the conditions at transitions. Thus, the LWR model can only characterize traffic moving with the equilibrium velocity distribution whereas the proposed model also considers the traffic conditions at transitions such as the safe velocity and safe time. These are important parameters and so the proposed model provides a more realistic characterization of traffic behaviour.

5. Conclusion

The LWR model only considers homogeneous traffic flow conditions. Thus, it cannot characterize variations in the flow, in particular transitions when abrupt changes in the density occur. To overcome this drawback, a model was developed which incorporates changes in the velocity during transitions based on the safe time and safe distance. The LWR and proposed models were evaluated under different traffic conditions and with two equilibrium velocity distributions. The results obtained show that the proposed model provides a more realistic characterization of traffic behaviour. Thus, it is a better choice for traffic management to reduce fuel consumption and improve public safety and air quality. The proposed model can be employed in connected vehicles and roadside units to mitigate traffic congestion. It can be extended by including lateral and forward distance headways to improve traffic flow prediction.

6. Declarations

6.1. Author Contributions

Conceptualization, Z.H.K. and T.A.G.; methodology, Z.H.K.; software, Z.H.K.; validation, Z.H.K. and T.A.G.; formal analysis, Z.H.K.; investigation, Z.H.K.; writing original draft, Z.H.K.; writing review and editing, Z.H.K., T.A.G., and W.I.; supervision, T.A.G.; project administration, T.A.G.; funding acquisition, T.A.G. All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

No new data were created or analyzed in this study. Data sharing is not applicable to this article.

6.3. Funding and Acknowledgements

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6.4. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Curve Number Estimation for Ungauged Watershed in Semi-Arid Region

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Abstract

The Benanain Watershed is located in East Nusa Tenggara with an area of 3,181 km² and is divided into 29 sub-watersheds. The East Nusa Tenggara itself is an eastern region of Indonesia with a unique climate condition called semi-arid. The high rainfall intensity occurring in short duration results in large surface runoff and erosion. Floods and erosion in semi-arid areas due to sensitive soils to drought and heavy rainfall extremely. This paper presents the application of the Soil Conservation Services-Curve Number (SCS-CN) real-flood flows through a digital map of soil type, land use, topography, and the heterogeneity of physical condition, especially for ungauged watersheds. The method used is an approach empirical to estimate runoff from the relationship between rainfall, land use, and soil hydrology groups. This watershed has a large area that must analyze every sub-watershed. The land-use of the Benanain watershed is secondary dryland forest by 44.26% and the hydrological soil group on the B group classification with medium to high absorption potential by 46.502% from the total area. The curve number value of the Benanain Watershed ranges from 56.54 to 73.90, where the mean CN value of 65.32. The rainfall (mm) for the 29 sub-watersheds in the Benanain Watershed has decreased by about 74.65% when being surface runoff or only 25.35% of water becomes surface runoff. The relationship between rainfall depth and CN is classified as standard response and trend line (flat slope) equilibrium occurs when rainfall depth value of 56.71 mm and CN is close to 66.30. The high variability of intense rainfall between the rainy season and the dry season had a significant influence on the curve number value in a large watershed area. Further analysis will be more accurate if it is supported by long rainfall data and observation runoff data as a control.

Keywords: SCS-CN Method; Soil Type; Land Use; Standard Response.

1. Introduction

A semi-arid region is an area that receives lower rainfall compared to potential evapotranspiration. This semi-arid region covers 31% of the world's [1]. The climate in East Nusa Tenggara (NTT) is of type D with the rainy season (average of monthly rainfall > 200 mm) that occurred between 3-4 months [2]. The characteristic of semi-arid regions is the evapotranspiration is much greater than precipitation, and the intensity of rainfall is very high during the rainy season. High-intensity rainfall variability during the rainy season and dry season causes enormous erosion [3, 4]. The land surface temperature affected land use, land cover, vegetated areas, water resources, etc. [5]. East Nusa Tenggara which has semi-arid conditions, is of course also affected by the land surface temperature.
The large volume of rainfall exceeds the soil infiltration capacity resulting in a large of runoff and erosion. These extreme climatic conditions often lead to misinterpretation of rainfall data, especially in East Nusa Tenggara. That is because the rainfall does not spread evenly throughout the year [6]. The geographical form as thought watershed shape also has a big influence on rainfall. Large local storms can shed a large part of annual rain in just a few days or hours. That causes the river to experience massive flooding every year.

Flood modeling design is inseparable from the rainfall-runoff relationship, and one method that can use is the Natural Resources Conservation Service Curve Number (NRCS-CN). The advantage of this method was can used in areas where flood hydrograph data or automatic recording of water levels are not available; however, rainfall recording data are available [7]. Also, this NRCS-CN model selection be affected by several factors, such as: (1) a familiar model is used over the years around the world, (2) very efficient, (3) the required data input generally available, and (4) this model connecting runoff to soil type, land use and control practices [8].

NRCS-CN has several main elements in the rainfall-runoff process, such as watershed boundaries, rainfall, hydrological abstraction, and runoff [9]. Determination of curve number for watersheds characteristics that do not have flood hydrograph data and water level reservoir also can be seen from the watershed, namely soil type, land use, conditions hydrology, and antecedent moisture condition (AMC). NRCS-CN water loss method and the NRCS unit hydrograph also used to determine hydrologic soil groups from the Harmonized World Soil Database (HWSD) map. In this method, the runoff thickness or rainfall is a function of the total rainfall thickness and reflection parameter of the number of runoff curves called Curve Number [10].

Soil Conservation Services and Curve Number (SCS-CN) technique, also known as the Natural Resources Conservation Service Curve Number (NRCS-CN) is one of the simplest, and well-documented conceptual methods to predict rainfall-runoff. The SCS-CN model application was to estimate runoff from small watersheds and runoff processes typical of a watershed-based on a remote sensing geo-information [11]. The determination of the correction curve number value has been done in an oval watershed using the HEC-HMS model and influencing factors such as soil properties, geological formation, and land use [12].

The SCS-CN application in the Temef watershed shows that the soil hydrological group affects the flood water level [13]. The effect of river morphometric parameters on the potential for runoff in an ungauged river using satellite imagery, topographic maps, and rainfall data combined with geospatial techniques have been done by the previous [14]. The CN value can be used for comparative studies of the impact of urbanization and forest fires and their combined effect on the runoff response [15]. Estimating the curve number value use the SCS-CN method has been carried out by previous researchers because of its simplicity and practical design [18-20]. At present, many researchers use the GIS and Remote Sensing techniques to explore and analyze the curve number method in a small watershed [16, 17, 21]. The CN values also could be verified using a rainfall simulator confirmed by the statistical tests [22, 23].

This study aims to determine a Curve Number (CN) value based on a hydrogeological map diversified to the hydrologic soil group map and check its applicability to the ungauged watershed such as in the Benanain Watershed. The characteristics of the Benanain watershed have very extreme fluctuation discharge, which indicates that the watershed is experiencing critical damage. Soil physical properties and land cover characteristics in semi-arid areas affect the value of curve numbers such as the Benanain watershed.

2. Research Methodology

2.1. Soil Conservation Service-Curve Number (SCS-CN) Method

The Soil Conservation Services (SCS) method was developed from rainfall observation over many years and has involved many agricultural areas in the United States. This method attempts to relate watershed characteristics such as soil, vegetation, and land use with the curve number (CN), which shows the potential flow for a particular rainfall.

The CN method has based on the relationship between the infiltration of each soil type and the amount of rainfall that falls every time it rains. The CN values range between 1 and 100 that is a function of runoff resulting from soil types, land use, hydrological conditions, and antecedent moisture condition [10, 16].

The form of the equation is:

\[ Q = \frac{(P-I_a)^2}{P-I_s+S} \]  

(1)

Where \( Q \) = direct runoff (mm), \( P \) = rainfall depth/precipitation (mm), \( I_a \) = initial abstraction (Initial loss), and \( S \) = the water maximum retention potential by the soil, which a big part because of infiltration (mm).

The definition of initial loss is part of the rainfall used to wet the soil surface include plants, trash, and vacant land before the infiltration happened. For impermeable surface or cement coated, the initial loss is the amount needed to moisten the surface before water accumulates and becomes runoff. For forest land cover, lose initial is usually taken
5.1 - 13 mm needed to wet the surface. Total this is rarely measured, and the SCS assumes that the initial loss is approaching 0.2 times the maximum soil moisture retention or S.

Woodward et al. used 307 watersheds scattered in America, which has more than 20 flood events [24]. The results indicated a $\lambda$ value of about 0.05 gives a better fit to the data and would be more appropriate for using the runoff simulations. That $\lambda$ is not constant from watershed to watershed, and the assumption of $\lambda = 0.20$ is unusually high.

In determining the depth of excess rainfall or surface runoff, the correlation between $I_a$ and S values shown as follows [7, 17]:

$$I_a = \lambda \cdot S$$  \hspace{1cm} (2)

$$I_a = 0.2 \cdot S$$  \hspace{1cm} (3)

Based on Equation 3, the value of runoff depth (mm) is used in the following formula [20, 25]:

$$Q = \frac{(P-I_a)^2}{(P+0.05S)} \text{ for } P \geq 0.2$$  \hspace{1cm} (4)

$$Q = 0 \text{ for } P \leq 0.2 \cdot S$$  \hspace{1cm} (5)

The maximum retention (S), and characteristics of the watershed associated with the intermediate parameter, namely Curve Number (CN).

$$S = \frac{25400 - 254 \cdot CN}{CN}$$  \hspace{1cm} (6)

Where CN (Curve Number) is a representation of potential runoff from the land cover – soil complex characteristics [10]. For watersheds with sub-watersheds that have a different land type and land cover, then CN composite values are determined based on:

$$CN = \frac{CN_1 \cdot A_1 + CN_2 \cdot A_2 + \ldots + CN_n \cdot A_n}{\sum_{i=1}^{n} A_i}$$  \hspace{1cm} (7)

Where $CN_i$ is the CN value in the sub-watershed $i$, $A_i$ is the area in sub-watershed $i$, and $n$ is the number sub-watershed. A combination of a hydrologic soil group, land use, and treatment class can be shown in Table 1 [8].

To estimate the curve number from rainfall-runoff data for S as a function of precipitation and depth runoff (Q) forms the equation [25]:

$$S = 5 \left( P + 2Q - \sqrt{(4Q^2 + 5PQ)} \right)$$  \hspace{1cm} (8)

The strong relationship between CN and rainfall depth occurred when Equations 5 and 7 are used to calculate values of CN from observed rainfall depth and runoff depth. The CN method is often used as a transformation of design rainfall depth to design runoff depth for a given return period [25]. To determine the curve number for rainfall depth and runoff depth has been applied for an ungauged watershed in the Benanain River.

<table>
<thead>
<tr>
<th>No.</th>
<th>Land Covering</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Airport</td>
<td>79</td>
<td>86</td>
<td>90</td>
<td>92</td>
</tr>
<tr>
<td>2</td>
<td>Marshy Bush</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
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<tr>
<td>3</td>
<td>Primary Dryland Forest</td>
<td>25</td>
<td>55</td>
<td>70</td>
<td>77</td>
</tr>
<tr>
<td>4</td>
<td>Secondary Dryland Forest</td>
<td>25</td>
<td>55</td>
<td>70</td>
<td>77</td>
</tr>
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<td>Primary Mangrove Forest</td>
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</table>
Soil Conservation Services (SCS) has developed a soil classification system based on soil properties and classified it into four hydrological soil groups (Table 2). Soil Conservation Services has developed a soil classification system based on soil properties and classified it into four hydrological soil groups (Table 2). It is ranging from soil type A (very absorb water), B (potency absorbs moderately water), C (potency absorbs less water), and D (potency absorbs the least water). The definitions for each soil group adjusted by looking at the similarities to the potential surface runoff under the same weather conditions and land use.

<table>
<thead>
<tr>
<th>No.</th>
<th>Land Covering</th>
<th>A</th>
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<tr>
<td>20</td>
<td>Open Land</td>
<td>30</td>
<td>58</td>
<td>71</td>
<td>78</td>
</tr>
<tr>
<td>21</td>
<td>Transmigration</td>
<td>59</td>
<td>74</td>
<td>82</td>
<td>86</td>
</tr>
<tr>
<td>22</td>
<td>Body of Water</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 2. Hydrologic soil group classification

<table>
<thead>
<tr>
<th>Land Group</th>
<th>Information</th>
<th>Infiltration Rate (mm/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>The smallest running water potential. Including deep sand soil with elements of dust and clay.</td>
<td>8-12</td>
</tr>
<tr>
<td>B</td>
<td>Water potential. Small runoff, sandy soil shallower than A. fine to medium texture, medium infiltration rate.</td>
<td>4-8</td>
</tr>
<tr>
<td>C</td>
<td>Medium runoff water potential. Shallow soil and contains enough clay. Medium to smooth texture. Low infiltration rate.</td>
<td>1-4</td>
</tr>
<tr>
<td>D</td>
<td>High Runoff Water Potential, mostly clayey, shallow, with an impermeable layer near the soil surface. Very low infiltration.</td>
<td>0-1</td>
</tr>
</tbody>
</table>

The depth and hydraulic conductivity of any water-impermeable layer and the depth to any high-water table, it used to determine the correct hydrologic soil group for the soil. Hydraulic conductivity is a quantitative measure of the ease with which water transmits through soil pores depending on the rock permeability. Lithological composition of rocks and permeability conditions to give qualitative information on the soil permeability, the occurrence of groundwater, and productivity of aquifers. It provides an overview of the depth of the aquifer (Table 3). Therefore, these two parameters are interrelated and linear relationship (the increases the hydraulic conductivity, the greater the permeability).

Table 3. Lithological composition of rocks and permeability conditions

<table>
<thead>
<tr>
<th>No.</th>
<th>Figure</th>
<th>Lithological and Permeabilities</th>
<th>Aquifers</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td><img src="image1.png" alt="Image" /></td>
<td>Pillow lava, generally low permeability (Fissures and Porous) of Poor productivity Regions without exploitable: groundwater and regions without exploitable groundwater</td>
<td>Aquifers in which flow is intergranular: Extensive, productive aquifer (Aquifer of moderate transmissivity; water table generally above or near the surface; wells yield generally more than 5 l/sec)</td>
</tr>
<tr>
<td>2</td>
<td><img src="image2.png" alt="Image" /></td>
<td>The alluvium is composed of sands, gravels, cobbles, clays, and mud. Moderate to high permeability in coarse materials; low permeability in fine materials.</td>
<td>Aquifers in which flow is through fissures, fractures, and channels: Moderately productive aquifers.</td>
</tr>
<tr>
<td>3</td>
<td><img src="image3.png" alt="Image" /></td>
<td>Sandy marl interbedded with sandstone, conglomerate, and dactitic tuff. Generally low to moderate permeability. (Fissures and Porous) of Poor productivity Regions without exploitable: Poorly productive aquifers of local importance</td>
<td>Aquifers in which flow is through fissures, fractures, and channels: Moderately productive aquifers.</td>
</tr>
<tr>
<td>4</td>
<td><img src="image4.png" alt="Image" /></td>
<td>Predominantly consist of massive limestone and calcilutite. Low to moderate permeability, depends on the degree of fissuration.</td>
<td>Aquifers in which flow is through fissures, fractures, and channels: Moderately productive aquifers.</td>
</tr>
<tr>
<td>5</td>
<td><img src="image5.png" alt="Image" /></td>
<td>Generally, coralline limestone, locally karstified. Permeability varies, depends on the karstified degree.</td>
<td>Aquifers in which flow is through fissures, fractures, and channels: Moderately productive aquifers.</td>
</tr>
</tbody>
</table>

Soil permeability was determined using the conversion of hydrogeological maps through to the Hydrological Soil Group maps [7] are shown in Table 4 and Table 5.
Table 4. Transfer of hydrogeological map to HSG for deep groundwater, more than 100 cm

<table>
<thead>
<tr>
<th>Permeability</th>
<th>Very High</th>
<th>High</th>
<th>Moderate</th>
<th>Low</th>
<th>Very Low</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>D</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5. Transfer of the hydrogeological map to HSG for shallow groundwater, less than 100 cm

<table>
<thead>
<tr>
<th>Permeability</th>
<th>Very High</th>
<th>High</th>
<th>Moderate</th>
<th>Low</th>
<th>Very Low</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>D</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.2. Study Area and Data Used

Benanain River is the main river of the Benanain Watershed which has an area of 3,182 km², where the Benanain River flows across four districts, namely Malaka District, Belu District, North Central Timor District, and South-Central Timor District. It is located at approximately 124°11’45.64” - 125°07’31.22” E and 8° 56’33.21” - 9° 58’34.60” S (Figure 1).

Malaka district is approximately 232 km from Kupang City to the east of Timor Island. This research was conducted in the Benanain watershed that flows to the estuary in the southeast of the Timor Sea. Figure 1 shows the changes in the shape of the river flow where there is a meander phenomenon. The river morphology has changed a lot. It has happened as a result of riverbank erosion which causes silting in the river channel. This sedimentation makes things worse if the flood intensity is high enough because the river cross-section was unable to accommodate the water flow.

The Malaka district has a tropical climate with an average temperature of 24 - 34 °C. The maximum daily rainfall has varied between 16-68 mm in the eastern region, while 120-255 mm in most of the northern. The average annual rainfall has estimated at 1,500 mm/year [26]. This research used ten rainfall stations for ten years (1996-2008) and...
spread across four districts that crossed by the Benanain watershed. The average areal rainfall for the analyzed part of the catchment was determined using the Thiessen polygon method, as showed in Figure 2. The geographical location of rainfall stations shown in Table 6.

Table 6. The geographical location of rainfall stations in Benanain watershed

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Station Code</th>
<th>Latitude</th>
<th>Longitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sukabitetek</td>
<td>009 TTS</td>
<td>9°18’56.9”</td>
<td>125°51’15.6”</td>
</tr>
<tr>
<td>Uabau</td>
<td>018 KUP</td>
<td>9°6.3’3”</td>
<td>124°23’3”</td>
</tr>
<tr>
<td>Fatumnasi</td>
<td>006 KUP</td>
<td>9°38’53.7”</td>
<td>124°13’29.3”</td>
</tr>
<tr>
<td>Noenoni</td>
<td>024 TTS</td>
<td>9°31’48”</td>
<td>124°18’0”</td>
</tr>
<tr>
<td>Oeoh</td>
<td>011 TTU</td>
<td>9°7.16’7”</td>
<td>124°4.66’7”</td>
</tr>
<tr>
<td>Kefamenanu</td>
<td>009 TTS</td>
<td>9°39’43.81”</td>
<td>123°59’6.36”</td>
</tr>
<tr>
<td>Noemuti</td>
<td>008 TTS</td>
<td>9°35’30.39”</td>
<td>124°28’48.56”</td>
</tr>
<tr>
<td>Fatuhao</td>
<td></td>
<td>9°42’24.7”</td>
<td>124°34’23.4”</td>
</tr>
<tr>
<td>Sekon</td>
<td></td>
<td>9°27’37.2”</td>
<td>124°38’3.5”</td>
</tr>
<tr>
<td>Oenopu</td>
<td></td>
<td>9°42’18.94”</td>
<td>123°57’33.2”</td>
</tr>
</tbody>
</table>

Land use activities in the Benanain watershed cause a change in the type of land cover, changes in vegetation, deforestation, shifting cultivation, converting forests to plantations, and changes in land management. The overflow in the Benanain River occurs almost every year and even repeatedly in a year. The flood tends to increase depending on the intensity of rainfall that occurs upstream. Therefore, it is necessary to the analysis of effects of land use and soil types on the curve number (CN) as one of the determining variables for discharge changes in the Benanain watershed. The characteristic of ten rainfall-runoff direct events for the analysis, with the calculated curve number according to Equations 4 and 6 presented in Table 7.

Table 7. Characteristics data in the Benanain watershed

<table>
<thead>
<tr>
<th>Category</th>
<th>Unit</th>
<th>Value for the events average</th>
<th>range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall depth/precipitation</td>
<td>mm</td>
<td>82.84</td>
<td>56.71 - 130.27</td>
</tr>
<tr>
<td>Maximum retention (S)</td>
<td>-</td>
<td>137.22</td>
<td>89.71 - 195.24</td>
</tr>
<tr>
<td>Runoff depth (Q)</td>
<td>mm</td>
<td>17.19</td>
<td>4.41 - 51.67</td>
</tr>
<tr>
<td>Curve Number</td>
<td>-</td>
<td>65.32</td>
<td>56.53 - 73.90</td>
</tr>
</tbody>
</table>
The research flow chart to estimate the curve number model and to predict the amount of surface runoff is presented in Figure 3. The amount of runoff depth obtained in this study includes a rainfall depth (P) and the curve number prediction. Because the catchment area doesn't have observed runoff data, this study only estimates the curve number data from land use maps and soil maps. Furthermore, to determine the correlation between P and Q, Q and CN were done using statistical analysis.

3. Result and Discussions

3.1. Hydrological Soil Group Map

The Benanain watershed area has divided into 29 sub-watersheds (Figure 4), and based on the land use map for the island of Timor, the land cover types of the Benanain watershed can classify as in Table 8.
It can be seen in Table 8 that the Benanain watershed has dominated by secondary dryland forest covering an area of 1,408.20 km² with CN values based on land use A = 25, B = 55, C = 70, and D = 77. Secondary dryland land forest in the Benanain watershed covers 44.26% of the total area, which means that is better than scrub or savanna. The secondary dryland forest has granular aggregates that are better at absorbing water. It has a scaly clay soil texture, moderate soil permeability, very poor soil porosity, moderately content weight, and low soil organic level content. The clay particles are more difficult to detach than sand or gravel, but clay is easier to transport.

<table>
<thead>
<tr>
<th>No.</th>
<th>Land Use</th>
<th>Area (Km²)</th>
<th>Percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Secondary dryland forest</td>
<td>1,408.20</td>
<td>44.26</td>
</tr>
<tr>
<td>2</td>
<td>Shrubs</td>
<td>716.84</td>
<td>22.53</td>
</tr>
<tr>
<td>3</td>
<td>Habitation</td>
<td>20.93</td>
<td>0.66</td>
</tr>
<tr>
<td>4</td>
<td>Savanna</td>
<td>663.22</td>
<td>20.85</td>
</tr>
<tr>
<td>5</td>
<td>Open land</td>
<td>31.72</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>Primary dryland forest</td>
<td>100.17</td>
<td>3.15</td>
</tr>
<tr>
<td>7</td>
<td>Dryland farming mixed with bush</td>
<td>68.26</td>
<td>2.15</td>
</tr>
<tr>
<td>8</td>
<td>Body of water</td>
<td>0.25</td>
<td>0.01</td>
</tr>
<tr>
<td>9</td>
<td>Dryland farming</td>
<td>171.94</td>
<td>5.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3,181.521</td>
<td>100.00</td>
</tr>
</tbody>
</table>

Table 7. Percentage of land use types in the Benanain watershed

Hydrologic soil group (HSG) is determined by the water transmitting from the soil layer with the lowest saturated hydraulic conductivity and value of depth to the impermeable layer or a water table. A hydrogeological map gives complete information on the parameters needed for HSG determination as soil permeability, groundwater level position from the surface can use as the basis to determine the HSG value shown in Tables 4 and 5. Digitalization using ArcGIS based on the hydrogeological map of Timor Island obtained soil hydrological types and the groundwater level depth which divided into 19-types with low to very low, low to moderate, and moderate to high passing types with HSG B, C, and D (Table 9). Determination of the Curve Number Value in the Benanain watershed using the SCS-CN method needs to be done by overlying (intersection) the hydrological soil group map and land use map. The results of intersection reclassification of land use maps and soil hydrology group maps are new polygons that represent the value of the curve number from the SCS. The result of the curve number value for 29 sub-watersheds in the Benanain watershed has shown in Table 9.

<table>
<thead>
<tr>
<th>No.</th>
<th>Lithology</th>
<th>Permeability</th>
<th>HSG</th>
<th>Area (Km²)</th>
<th>Percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Calciulite and marl</td>
<td>Low to very low</td>
<td>D</td>
<td>26.564</td>
<td>0.835</td>
</tr>
<tr>
<td>2</td>
<td>Conglomerates and crusts, are loose at the top and solid at the bottom</td>
<td>Moderate to high</td>
<td>C</td>
<td>8.869</td>
<td>0.279</td>
</tr>
<tr>
<td>3</td>
<td>Calciulite and marl</td>
<td>Low to moderate</td>
<td>C</td>
<td>3.368</td>
<td>0.106</td>
</tr>
<tr>
<td>4</td>
<td>Lava pillow</td>
<td>Low to very low</td>
<td>D</td>
<td>33.895</td>
<td>1.065</td>
</tr>
<tr>
<td>5</td>
<td>Conglomerates and limestone</td>
<td>Moderate to high</td>
<td>C</td>
<td>18.559</td>
<td>0.583</td>
</tr>
<tr>
<td>6</td>
<td>Alluvium consists of sand, gravel, gravel, clay, and mud</td>
<td>Low to moderate</td>
<td>D</td>
<td>35.514</td>
<td>1.116</td>
</tr>
<tr>
<td>7</td>
<td>Alluvium consists of sand, gravel, gravel, clay, and mud</td>
<td>Moderate to high</td>
<td>B</td>
<td>50.846</td>
<td>1.598</td>
</tr>
<tr>
<td>8</td>
<td>Conglomerates and crusts, are loose at the top and solid at the bottom</td>
<td>Moderate to high</td>
<td>B</td>
<td>140.021</td>
<td>4.401</td>
</tr>
<tr>
<td>9</td>
<td>Conglomerates and limestone</td>
<td>Low to moderate</td>
<td>C</td>
<td>41.916</td>
<td>1.317</td>
</tr>
<tr>
<td>10</td>
<td>Coral limestone, localized</td>
<td>Moderate to high</td>
<td>B</td>
<td>69.065</td>
<td>2.171</td>
</tr>
<tr>
<td>11</td>
<td>Alluvium consists of sand, gravel, gravel, loam, and sand</td>
<td>Moderate to low</td>
<td>C</td>
<td>62.677</td>
<td>1.970</td>
</tr>
<tr>
<td>12</td>
<td>Conglomerates and crusts, are loose at the top and solid at the bottom</td>
<td>Moderate to high</td>
<td>C</td>
<td>482.616</td>
<td>15.169</td>
</tr>
<tr>
<td>13</td>
<td>Coral limestone, localized</td>
<td>Moderate to high</td>
<td>B</td>
<td>420.485</td>
<td>13.216</td>
</tr>
<tr>
<td>14</td>
<td>Sandstone marl interspersed with sandstone, conglomerates, and dacitic tuffs</td>
<td>Moderate to high</td>
<td>B</td>
<td>537.489</td>
<td>16.894</td>
</tr>
<tr>
<td>15</td>
<td>Solid limestone and calcitrite</td>
<td>Moderate to high</td>
<td>B</td>
<td>224.524</td>
<td>7.057</td>
</tr>
<tr>
<td>16</td>
<td>The scaly clays contain chunks of other rock</td>
<td>Low to very low</td>
<td>D</td>
<td>816.918</td>
<td>25.677</td>
</tr>
<tr>
<td>17</td>
<td>Basalt lherzolite and serpentinite</td>
<td>Low to moderate</td>
<td>C</td>
<td>23.736</td>
<td>0.746</td>
</tr>
<tr>
<td>18</td>
<td>Various types of metamorphic rocks from basalt to genes, amphibolite, quartzite, and granulite</td>
<td>Low to moderate</td>
<td>C</td>
<td>147.787</td>
<td>4.645</td>
</tr>
<tr>
<td>19</td>
<td>Solid volcanic breccias, agglomerates, lava and tuffs</td>
<td>Moderate to high</td>
<td>B</td>
<td>36.672</td>
<td>1.153</td>
</tr>
</tbody>
</table>

| Total |                                  | 3181.52     | 100.00        |
The percentage of each soil type is tabulated in Table 8 to determine the dominance of soil types in the Benanain watershed. The sub-basin area that is classified as low to lowest (Type D) is 28.683%, low to moderate (Type C) is 24.815%, and the remaining moderate to high (Type B) is 46.502%. The lithology of soil types with the highest percentage is scaly clay containing boulders of other rock with low to very low permeability (the potential for water absorption is the least) has a value of 25.677%.

However, it can be seen in Table 9, based on the hydrologic soil group, the B group classification with medium to high absorption potential has a large area. It means water will absorb into the soil surface as infiltration by 46.502% of the watershed area when it rains. The classifications of hydrologic soil group in Benanain watershed with type C and D, which have medium to very low permeability indicate that most of the Benanain watershed has a potential for surface runoff. Soils in this group (Type C/D) have moderately to high runoff potential when thoroughly wet. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 % clay, less than 50 % sand, and have clayey textures.

The CN value of the Benanain watershed ranges from 56.55 to 73.90 and the average value of 65.32, and the highest CN value found in Sub-watershed W-310, which dominated by the type of scaly clay containing boulders of other rock and land cover in the form of shrub bush with an area of 22.61 km² or 0.21%. The curve number and hydrologic soil group values of the Benanain watershed depicted on maps (Figure 5).

3.2. Estimating Rainfall-Runoff

The runoff depth (Q) can be estimated using Equation 4 from the annual maximum daily rainfall data of 29 sub-watersheds in the Benanain watershed presented in Table 10.

Table 10. The Result of runoff calculation using the Curve Number method

<table>
<thead>
<tr>
<th>No.</th>
<th>Sub-Watershed</th>
<th>Area (km²)</th>
<th>CN</th>
<th>S</th>
<th>P   (mm)</th>
<th>Q   (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W300</td>
<td>102.02</td>
<td>69.84</td>
<td>109.70</td>
<td>62.00</td>
<td>10.72</td>
</tr>
<tr>
<td>2</td>
<td>W310</td>
<td>110.35</td>
<td>73.90</td>
<td>89.71</td>
<td>84.60</td>
<td>28.42</td>
</tr>
<tr>
<td>3</td>
<td>W320</td>
<td>278.74</td>
<td>67.48</td>
<td>122.42</td>
<td>56.71</td>
<td>6.72</td>
</tr>
<tr>
<td>4</td>
<td>W330</td>
<td>121.4</td>
<td>71.72</td>
<td>100.17</td>
<td>86.72</td>
<td>26.65</td>
</tr>
<tr>
<td>5</td>
<td>W340</td>
<td>103.58</td>
<td>70.51</td>
<td>106.24</td>
<td>64.34</td>
<td>12.43</td>
</tr>
<tr>
<td>6</td>
<td>W350</td>
<td>83.78</td>
<td>68.78</td>
<td>115.28</td>
<td>130.28</td>
<td>51.67</td>
</tr>
<tr>
<td>7</td>
<td>W360</td>
<td>150.13</td>
<td>66.19</td>
<td>129.74</td>
<td>72.90</td>
<td>12.48</td>
</tr>
<tr>
<td>8</td>
<td>W370</td>
<td>296.11</td>
<td>71.40</td>
<td>101.76</td>
<td>82.22</td>
<td>23.39</td>
</tr>
<tr>
<td>9</td>
<td>W380</td>
<td>31.44</td>
<td>56.54</td>
<td>195.25</td>
<td>70.69</td>
<td>4.41</td>
</tr>
<tr>
<td>10</td>
<td>W390</td>
<td>51.06</td>
<td>67.39</td>
<td>122.89</td>
<td>65.26</td>
<td>10.12</td>
</tr>
</tbody>
</table>
Model validation should be done to examine the accuracy of the SCS Curve Number Method. However, because there is no observational runoff data in the study area, the design runoff can be assumed from the empirical relationship between the rainfall depth, curve number, and runoff depth. The relationship between rainfall depth (P) and runoff depth for the average of curve number 65.32 and has overlaid with the graphical solution of Equation 6 shown in Figure 6 [27].

\[ Q = f(P) = 0.006 P^2 - 0.5314 P + 19.066 \]  
(9)

With \( r^2 = 0.683 \) of CN = 65.32. Based on Equation 9, the increase in rainfall depth value by 7.03% will also affect the
increase in runoff depth by 8.68% for CN = 65.32 in the Benanain watershed. Dots distribution shown in Figure 6 indicates a strong secondary relationship between rainfall depth vs. runoff depth for the mean of CN = 65.32. Hawkins [29] proposed to use an asymptotic function for the approximation of the relationship CN vs. rainfall depth values with the following equation:

$$CN = CN_\infty + (100 - CN_\infty) \exp(-kp)$$ \hspace{1cm} (10)

Where $CN_\infty$ is the constant values as $P \to \infty$; and $k = \text{fitting constant}$. Equation 10 was fitted using a least-square procedure for $CN_\infty$ and $k$. The fitting “k” can be following the equation:

$$CN = CN_\infty + (100 - CN_\infty) = 100 \frac{(2 + kP)}{(2+P)}$$ \hspace{1cm} (11)

Where $CN_\infty = 100/ (1 + P/2)$ for $P \to \infty$ then $CN(P) \to 100 \ k = CN_\infty$

A standard asymptote occurs if there is a tendency for CN to decrease and then approach a constant value with increasing rainfall depth (mm) [25, 28]. A similar trend was found for rainfall depth events and respective CN values as shown in Figure 7, considering $P > 0.2 \ S$ and $CN_\infty = 100/ (1 + 50.8 \ P)$.

Figure 7 shows a tendency for CN to decrease and then approach a constant value with increasing rainfall depth (mm) according to the formula:

$$CN = f(P) = f (77.67 P^{-0.04})$$ \hspace{1cm} (12)

Where the values of the curve number are estimated from 56.54 -73.90 with the mean value equals 65.32. As there is a tendency for curve numbers to decrease with the increase of rainfall depth. Therefore, in the application of mean CN (65.32) for design floods estimation is not allowed. The trend line obtained on the Benanain watershed is similar to previous researchers [21, 26], but the area study (watershed area) is smaller than this case. Hawkins (1993) suggests that the asymptotic line is a common and standard hydrological response to watershed behavior [29]. Therefore, in the Benanain watershed, the relationship between rainfall depth and CN is classified as a standard response. The asymptotic approach has the advantages that the more efficient use of the available data sources may be applied to ungauged watersheds and the results are similar to those derived from original data [30].

The average rainfall depth (mm) for the 29 sub-watersheds in the Benanain watershed has decreased by about 74.65% when being surface runoff depth. The runoff depth of the Benanain watershed may also be affected by land-use of the watershed (secondary dryland forest), river slope (0.45%) into the watershed, shallow soils (0.56 m), drainage density, and the confluence of tributaries so that the estuary is like a bottleneck. The equilibrium of trend line (flat slope) when rainfall depth value of 56.71 mm and CN approaches 66.30. The rainfall-runoff value that takes into account the curve number value in the Benanain watershed is also one of the factors that influence the flooding that occurs. The CN value in the Benanain watershed is in AMC I during the dry season, but a significant change will occur during the rainy season, where the CN value in the watershed becomes AMC III. Determination of CN value with a large watershed area and the high variability of rainfall intensity has a significant effect on the CN value, which results in the amount of surface runoff.

Analysis of the value of the curve numbers in the ungauged watershed is very helpful to estimate the percentage of water infiltration and water flow as surface runoff. Analysis of large catchment areas such as the Benanain watershed required mapping in smaller sub-watershed areas so that prediction data could be obtained closer to real conditions in the field. Therefore, it is necessary to conduct a similar investigation of the small watershed, with long rainfall records and observed runoff data, so calibration values can be obtained to flood frequency analysis.
4. Conclusion

The hydrological soil types in the Benanain watershed has classified into 19-types with three levels of permeability, namely, moderate to high permeability, moderate to low permeability, and low to very low permeability consisting of HSG B (moderate water absorption potential), HSG C (less water absorption potential) and HSG D (very less water absorption potential). The land-use type in the Benanain watershed is secondary dryland land forest covers 44.26% of the total area, which means that is better at absorbing water than shrub or savanna. Based on the hydrologic soil group, the B group classification with medium to high absorption potential reaches 46.502% of the total area of the Benanain watershed. It means water will absorb into the soil surface as infiltration when it rains. The Benanain watershed is dominated by soil types with scaly clay lithology containing other rock blocks with low to very low permeability and grouped into HSG-D groups with an area percentage of 25.667%.

The curve number value of the Benanain watershed ranges from 56.55 to 73.90, where the highest curve number value is found in the W-310 sub-basin which is dominated by the type of scaly clay containing boulders of other rock. The land cover in the form of shrub agriculture with an area of 22.61 km² or 0.21%. The runoff depth value increased by 8.68% with increasing rainfall depth, and the increased runoff depth is directly proportional to the increased curve number value of 3.14%. Determination of CN value with a large watershed area and the high variability of rainfall intensity has a significant effect on the CN value in Benanain watershed. For further analysis with flood hydrograph using the HSS SCS method, it is necessary to have long rainfall data and observations runoff depth are required as calibration data for controlling the use of curve numbers.

5. Declarations

5.1. Author Contributions

D.S.K., W.B., and J.H.F. contributed to the conception, methodology, and research design of the study; Y.A.S. performed numerical studies and analyzed the data in the field; J.L. contributes guided, reviewed, and commented on the previous version of the manuscript; D.S.K and Y.A.S. wrote the first draft of the manuscript; D.S.K., W.B., J.H.F, and J.L. added review and editing in the discussion. All authors have read and agreed to the published version of the manuscript.

5.2. Data Availability Statement

The data presented in this study are available on request from the corresponding author.

5.3. Funding

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5.4. Acknowledgements

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5.5. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Experimental and Numerical Investigations on Flexural Behaviour of Prestressed Textile Reinforced Concrete Slabs

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Abstract

Nowadays, concrete is mostly prestressed with steel. But the application of prestressing steel is restricted in a highly corrosive environment area due to corrosion of prestressing steel, leading to a reduction in strength and may cause sudden failure. Carbon textile is considered an alternate material due to its corrosive resistance property, high tensile strength, and perfectly elastic. Prestressing is also the only realistic way to utilize fully ultra-high tensile strength in carbon textile material. In this study, experimental and numerical analyses were carried out for the flexural behaviour of prestressed and non- prestressed carbon textile reinforced concrete slabs. This study also focuses on the influences of textile reinforcement ratios, prestressing grades on the flexural behaviour of carbon textile reinforced concrete (TRC). Fifteen precast TRC slabs were tested, of which six were prestressed to various levels with carbon textile. The obtained results show that prestressing textile reinforcement results in a higher load-bearing capacity, stiffness, and crack resistance for TRC slabs. The first crack load of the prestressed specimens increased by about 85% compared with those of non-prestressed slabs. Three-dimensional finite element models were developed to provide a reliable estimation of global and local response. The modeling techniques accurately reproduced the experimental behaviour.

Keywords: Textile Reinforced Concrete; Prestressed Slabs; Carbon; Flexure.

1. Introduction

Textile reinforced concrete (TRC) is a new innovative material that uses mesh-like reinforcements combined with fine-grained concrete. Due to its non-corrosive textile reinforcement, made of alkali-resistant glass or carbon fibers, a minimum concrete cover is necessary to transfer bond stresses from the reinforcement to the concrete. High strength, ductility, and non-corrosiveness textile reinforcements make TRC very suitable for constructing thin-walled, lightweight shell structures [1]. The weight of textile concrete elements could be reduced by up to 45%, resulting in a reduced need for maternal resources. This material can also strengthen existing concrete structures as a repair layer and a protective function to prevent corrosion problems in RC structures [2]. Today, tensile strength up to 3000 MPa can be achieved depending on the fiber material. However, textile reinforcements' high strength property could not be effectively utilized due to concrete performances [3]. Concrete performs well under compression but will be easily cracked when subjected to tension, and reinforcement becomes mostly effective only after cracking occurs. In this case, the low reinforcement ratio and high strength of textile reinforcement, together with a lower modulus of elasticity, are disadvantageous for applying TRC structures. The first crack that forms becomes relatively wide, and subsequent cracks appear, resulting in a less stiff structure, and these may violate serviceability limit state requirements.

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In order to avoid or limit the cracks in concrete and, at the same time, utilize the actual tensile strength of the textile and make it act more actively, the TRC members should be prestressed. In the case of textile reinforcements, pre-tension prestressing may be the most appropriate method. This new approach is expected to increase the cracking load of TRC structures; thus, the uncracked state's serviceability is improved. On the other hand, the textile reinforcements are behaving linearly elastic until rupture. This property is particularly suitable for prestressing techniques. Prestressed TRC leads to more slender and thereby more economical and durable structural elements. Since TRC is still a relatively new construction material, which has not yet been standardized, little research was reported in this area, especially for the prestressed TRC structures. Krüger [4] conducted a study to investigate the bond behaviour of textiles used for prestressing in fine-grained concrete under different conditions. Following epoxy impregnation, the prestressing of the roving leads to enhanced bond strength and stiffness. Krüger also reported that using alkali-resistant glass textiles as a prestressing element is not practical due to creep and a low static fatigue limit. Zdanowicz [5] compared bond behaviour between prestressed specimens and non-prestressed control specimens and concluded that chemical prestressing positively influences the bond behaviour of concrete with textile reinforcement. In these tests, the chemically prestressed specimens reached 24% higher bond strength than non-prestressed ones.

Peled [6] found that pre-tensioning of fabrics and the time at which the tension is removed can significantly influence the composite's performance depending on yarn properties, mainly the viscous–elastic properties and fabric geometry. Yunxing [7] experimentally investigated the influences of textile layers, prestress levels, and short steel fibers on the tensile behaviour of carbon TRC. It was found that evident increases in first-crack stress and tensile strength were observed with increasing prestress grades. Therefore, the capacity of TRC composites at the serviceability state limit states can be upgraded by exerting a prestressing force on the textiles. Gopinath [8] carried out a study to determine the uniaxial tensile behaviour of prestressing TRC plate with two types of alkali-resistant glass textiles, woven and bonded, and their combinations. From this study, TRC with mechanically stretched textile exhibited lower crack width and average crack spacing than with manually stretched textile.

Yunxing et al. (2018) [9, 10] conducted experimental studies on the influences of the number of textile layers, prestress grades of textile on the flexural behaviour of basalt and carbon textile-reinforced concrete plate. The presence of prestressing or steel fibres improved first-crack and ultimate stresses of the TRC specimens. Compared with the first-crack stress, a more pronounced enhancement in the ultimate stress was achieved by adding steel fibres. Reinhardt et al. (2003) and Reinhardt & Krüger (2004) [11, 12] dealt with carbon and glass TRC slabs' flexural behaviour and reported that the impregnated carbon is very suitable for prestressing. The most considerable effect of prestressing is that the initial strain of fabric is anticipated and that deflection and crack width after first cracking is minimized. Meyer [13] studied the flexural behaviour of the prestressed aramid TRC, in which the textiles were evenly placed along with the thickness of the specimens. In their work, the prestress delayed the generation of cracks and improved the post-cracking flexural stiffness and bearing capacity of the specimens but decreased the ductility. Liu [14] investigated the flexural properties of basalt TRC slab with pre-tension, short carbon, steel, and AR-glass fibers through three-point bending in a utilizing drop-weight impact setup. In this research, the flexural impact responses of the basalt TRC with flexural strength, flexural modulus, maximum strain, and toughness were improved by pre-tension. Yunxing [15] also investigated the flexural behaviour of preloaded reinforced concrete beams strengthened with a prestressed carbon TRC plate under various preloading conditions. The test results showed that the prestressed carbon TRC plates could significantly increase the cracking and ultimate loads and effectively inhibit the propagation of cracks.

Despite those advantages, the number of practical applications using prestressed TRC is still negligible in the current state. One explanation for this situation is the available design codes and guides do not provide any recommendations for TRC since fundamental studies, and relevant applications are still limited. Although there are reference guidelines for determining the mechanical behaviour of TRC [16-19], there is no standard. Therefore, investigations are needed to characterize and understand the structural behaviour of the TRC prestressed slab. The overall objective of this research is to investigate the flexural performances of non-prestressed and prestressed TRC slabs. The influences of the number of textile layers, prestress grades on the flexural behaviour of carbon TRC slabs are also discussed in this paper. Based on the four-point bending tests, the load–deflection relationship, cracking mechanisms, and failure modes of each experimental case are presented and analyzed. Furthermore, experimental results are required to validate analytical models' ability to capture the critical response characteristics of the non-prestressed and prestressed TRC slabs.

The brief research methodology flow chart, as shown in Figure 1, has been adopted to achieve the study's objective. The following section will present the full description of the two phases of the experimental program. Phase 1 presents the detailed preparation of the material properties, including fine-grained concrete, carbon textile reinforcement, and bond strength between textile and fine-grained concrete. The 2nd phase is the laboratory experiment: casting two sets of specimens, test set-up, and analyzing the experimental results. The finite element modeling was continuously conducted to validate the experimental results.
2. Experimental Investigation

2.1. Specimens and Test Description

In this study, all the carbon TRC specimens had the same dimension of 900 mm (length) × 150 mm (width) × 40 mm (depth). Two sets of TRC specimens were tested, corresponding to the non-prestressed and prestressed slabs. In the first set, the TRC slabs use 1, 2, and 3 carbon textile layers, corresponding to the longitudinal reinforcement ratio of 0.34, 0.69, and 1.05%, respectively. In the 2nd set, six specimens were applied by prestressing force, corresponding to 50 and 70% tensile force of the textile layer in the uniaxial test. The initial prestressing values were selected based on the recommendations of conventional prestressed reinforced concrete theory. For each experimental case, three nominally identical specimens were manufactured. Specimens were named following the notation PxLyNz, where x = prestressed level in percentage, y = number of textile layers, and z = specimen number. For example, the label P50L1N2 represents the 2nd specimen with one layer of carbon textile applied by the prestressing level of 50%. The detailed information for each specimen is mentioned in Table 2.
Figure 2 shows the prestressing frame as prepared for a uniaxial prestressing of textile. The clamps and the hydraulic jack can be seen in the frame with a total size of about 2.0 × 0.5 m. The preparation process was initiated by fixing all the textiles on the prestressing frame at both ends of the device, and textiles were wrapped around two smooth rollers, which were then placed in the steel clamp. The pre-tension force was applied to the textile reinforcement by the hydraulic jack and was measured by the load cell fixed at the other end of the frame. Based on the material properties of carbon textile, the elongation of textile reinforcement was also measured to control the pre-tension force. After reaching the textiles’ prescribed prestress value, the chute at the tension end was fixed by the adjusting nut. Subsequently, fine-grained concrete was mixed and directly poured into the mold. Then, the mixture was vibrated thoroughly, and the top surface was smoothed by using a metal spatula. The TRC slabs were covered with wet cloths after the matrix was initially set. The prestressed ones were allowed to harden for 14 days before releasing the pre-tension force. All the slabs were then removed from their molds and cured until testing was performed after 28 days.

2.2. Material Properties

The fine-grained binder systems with a maximum grain size of 0.63 mm were designed explicitly for carbon textile application. The high-performance plasticizer and fly ash were added to achieve an excellent flowing capability of the concrete to ensure proper penetration of the fabrics’ small gaps. The fine-grained concrete was mechanically characterized by testing six 40×40×160 mm prisms. The average flexural strength and average compressive strength at 28 days were equal to 6.95 and 47.5 MPa, respectively. The average flexural strength and compressive strength at 14 days (at the time of releasing prestressed force) were identical to 5.72 and 38.7 MPa, respectively.

![Figure 3: Carbon textile and TRC specimen for the uniaxial tensile test](image)

In this study, the carbon textile reinforcement SITgrid017 was used. The carbon fiber yarns, having a count of 3200 tex, were processed in the warp and weft directions with a distance of approximately 12.7 mm between them. Each carbon roving consists of 48,000 fibers and has a cross-sectional area of 1.808 mm² (in both directions). A textile weighting unit area of roughly 578 g/m² was produced. The geometrical and mechanical characteristics are collected in Table 1. According to the Recommendation of RILEM TC 232-TDT [18], the tensile strength and elastic modulus of the fiber were measured through tensile tests on the uniaxial tensile specimens (Figure 3) and were equal to 2890 MPa, and 185 GPa, respectively.

<table>
<thead>
<tr>
<th>Geometric Characteristics</th>
<th>Mechanical Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roving distance (mm)</td>
<td>Tensile strength (MPa)</td>
</tr>
<tr>
<td>12.7 × 12.7</td>
<td>2890</td>
</tr>
<tr>
<td>Rovings/m</td>
<td>Elastic modulus (GPa)</td>
</tr>
<tr>
<td>78</td>
<td>185</td>
</tr>
<tr>
<td>Mesh size (mm)</td>
<td></td>
</tr>
<tr>
<td>10 × 8.5</td>
<td></td>
</tr>
<tr>
<td>Roving area (mm²)</td>
<td></td>
</tr>
<tr>
<td>1.808</td>
<td></td>
</tr>
</tbody>
</table>

2.3. Test Setup and Instrumentation

All the tests were conducted in the Structural Engineering Laboratory at the University of Transport and Communications, Vietnam. Specimens were monotonically loaded with four-points bending, using displacement controlled method, with a loading rate of 1 mm/min. The clear span of all the slabs was kept constant at 450 mm, and the shear span was 150 mm. The schematic view and a view of the test setup are shown in Figure 4. A linear variable
differential transformer (LVDT) was installed on the slab's bottom surface to measure its deflections during the test. Moreover, strain-gauges were used to record compressive and tensile strains of concrete at upper and lower surfaces during the experiment. A computer-based data acquisition system was used to record the load from the load-cell, the deflection from LVDT, and strain-gages.

2.4. Test Results

The load versus deflection curves for Set 1 and Set 2 are presented in Figures 5 and 7, respectively. Table 2 shows a summary of the load-deflection in cracking and ultimate of all test specimens. Besides, failure modes and crack patterns of tested slabs are illustrated in Figure 6 and Figure 8. The experimental results showed that the non-prestressed and prestressed slabs with one textile layer failed due to the carbon textile rupture (Figures 6-a and 8). Besides, the non-prestressed slabs with 2 and 3 textile layers failed due to the crushing of the concrete in the compression zone before rupture of the textile reinforcement (Figure 6-b).

Figure 5 compares the load-deflection of non-prestressed slabs with 1, 2, and 3 fabric layers. All slabs show similar load-bearing behaviour in the first stages. The load-deflection curves indicate a linear elastic behaviour, up to the point of first flexural crack appears, at a load level of 4 to 6 kN. The average cracking loads in 2 and 3-layers slabs are 14 and 18% higher than those of 1 layer slabs. The slabs' stiffness decreased after the first cracks, resulting in more significant deflection, especially in 1-layer slabs. Due to the bond between textile roving and fine-grained concrete, tensile stress was developed in the concrete until the fine-grained concrete's tensile strength is reached once more. With an increase of the tension force, an additional crack occurred in all types of tested slabs. After the cracks' appearance, the load-deflection curves showed small drops due to the transfer of tensile force from fine-grained concrete to textile reinforcement. The tensile stress in textile reinforcement will increase sharply, resulting in a significant increase in tensile strain. However, the slabs were loaded under displacement-control manner, the tensile strain in textile reinforcements could not reach the required level immediately. Hence, the applied load will decrease suddenly for a short while until the tensile stress increase again.
In 1-layer slabs, by a load increase, the rovings are strained up to their tensile strength. In this stage, the crack pattern was stabilized, no further cracks occur, but the most significant crack expanded larger. Then, the textile reinforcement was continuously broken, resulting in the flexural failure mode with rupture of textile reinforcement. It can be seen from Figure 5 that after cracking, the specimens with more textile layers had higher stiffness and hence had higher bearing capacity at the same mid-span deflection, which can also be proved by the results listed in Table 2. In contrast to 1-layer slabs, the non-prestressed slabs with 2 and 3 fabric layers exhibited the concrete's crushing in the compression zone at the deflection of around 7 mm. After the two and three-layers specimens reach their ultimate capacity (approximately 25.5 kN), their curves show a slight but constant decrease. This is due to the fact that the concrete could not carry the significant compression force, and the concrete zone fails more and more. This also means that the textiles' full tensile capacity cannot be used because of the weakness of the compression zone.

Table 2. Load, deflection, and modes of failure comparison for all specimens

<table>
<thead>
<tr>
<th>Set</th>
<th>Slab</th>
<th>Cracking</th>
<th>Ultimate</th>
<th>Failure mode</th>
<th>Energy dissipation (kN.mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Dc (mm)</td>
<td>Pcr (kN)</td>
<td>Dc (mm)</td>
<td>Pcr (kN)</td>
</tr>
<tr>
<td>Set 1: Non</td>
<td>P0L1N1</td>
<td>0.43</td>
<td>5.50</td>
<td>9.36</td>
<td>21.40</td>
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<td>pre-stress</td>
<td>P0L1N2</td>
<td>0.43</td>
<td>5.48</td>
<td>10.44</td>
<td>20.40</td>
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<tr>
<td></td>
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<td>4.97</td>
<td>10.34</td>
<td>19.75</td>
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<tr>
<td></td>
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<td>6.10</td>
<td>8.25</td>
<td>26.41</td>
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<tr>
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<td>8.17</td>
<td>25.38</td>
</tr>
<tr>
<td></td>
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<td>5.60</td>
<td>7.61</td>
<td>24.29</td>
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<tr>
<td></td>
<td>P0L3N1</td>
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<td>6.10</td>
<td>7.50</td>
<td>26.51</td>
</tr>
<tr>
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<td>0.38</td>
<td>6.37</td>
<td>6.28</td>
<td>25.88</td>
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<tr>
<td></td>
<td>P0L3N3</td>
<td>0.35</td>
<td>6.22</td>
<td>7.25</td>
<td>24.99</td>
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<tr>
<td>Set 2: Pre-</td>
<td>P50L1N1</td>
<td>0.65</td>
<td>9.89</td>
<td>6.32</td>
<td>22.40</td>
</tr>
<tr>
<td>stress</td>
<td>P50L1N2</td>
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<td>6.78</td>
<td>22.11</td>
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<tr>
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<td></td>
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<td>P70L1N2</td>
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<td>10.01</td>
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<tr>
<td></td>
<td>P70L1N3</td>
<td>0.53</td>
<td>9.44</td>
<td>6.05</td>
<td>21.55</td>
</tr>
</tbody>
</table>

Abbreviations:

- Dc: displacement at cracking.
- Pc: corresponding load at cracking.
- Pu: ultimate load at failure.
- Pu,avg: averaged ultimate load at failure.

Figure 6. Crack patterns of test specimens in Set 1
Figure 7 represents the influences of prestressing grades on the bending behaviour of TRC slabs. The behaviour of all prestressed specimens in Set 2 also presented a typical flexural failure mode, consisted of three stages, namely: (a) the un-cracked stage, (b) the cracked stage, and (c) the failure stage. The load was linear up to the first flexural crack initiation in pure bending span, followed by a non-linear behaviour up to failure. The average first-crack load of the prestressed specimens increased by 83.3 and 86.7%, respectively, compared with those of non-prestressed slabs. The compressive pre-strain in the fine-grained concrete induced by the prestressing force increased the first-crack stress and enhanced the flexural modulus. Since the carbon textile has no plastic capacity, the TRC specimens failed when the reinforcements reach their tensile strength. All textile rovings were continuously broken in a brittle manner. Before breaking, there was no sign of compressive failure in the top edge of TRC slabs. It should be noted that the effectiveness of prestressing force grades (i.e., 50 and 70%) in cracks resistance is not much different.

As shown in Table 2, prestressing forces on the textile also slightly improved the bearing capacity of the TRC specimens and reduced the ultimate deflection. This can be explained by the complexity of the tensile strength of textile rovings. The concrete cracks along the reinforcement lead to damage in the rovings and cause a decreasing strength of the component. The main effects responsible for this loss of strength are the lateral pressure and the bending stresses of the filaments at the crack edges [1]. As displayed in Figure 8, the crack's width in prestressed specimens before failure was much smaller than that in non-prestressed slabs. The smaller cracks resulted in the larger tensile strength of textile rovings. The comparison of data above indicated that the prestress on textile improved the first-crack and ultimate load capacities of the TRC specimens.
In order to apply TRC slabs in construction, the deflection of these structures is one of the checks that should be performed for serviceability limit state design. In this test, the clear span of all the slabs was kept constant at 450 mm. This research also compares the load capacity and the energy dissipation of prestressed and non-prestressed slabs at a deflection level of 2.25 mm, corresponding to 1/200 tested span. As shown in Figure 7 and Table 2, the average load in prestressed slabs is 85.6 and 91.2 % higher than those of non-prestressed slabs. Energy dissipation is estimated by the area under the load - deflection curves. The average energy in prestressed slabs is 92 and 97 % higher than those of non-prestressed slabs with 1 fabric layer. The energy in 50% prestressed slabs is also 44% and 48% higher than those of non-prestressed slabs with 2 and 3 fabric layers.

3. Numerical Investigation

3.1. Finite Modeling

The numerical analysis using ABAQUS 6.12-1 software was carried out to predict the ultimate capacity, the mechanical behaviour of the structures and compared to the measured results. A total of five FE models was performed for both prestressed and non-prestressed specimens. A full view of specimen P0L1-FEM is shown in Figure 9 for reference. Due to the symmetry of the specimen geometry and loading, only a quarter of the specimens were modelled to save the calculation time. The slab was restrained at the support employing hinges. The loading was applied continuously in the form of the displacement control manner.

![FE model of P0L1-FEM TRC slab](image)

Two components of the specimen (fine-grained concrete slab and textile reinforcement) were modelled separately and assembled to make a complete specimen model. Furthermore, element types, mesh sizes, boundary conditions, and load applications have been chosen so that the simulation results could agree with ones from experiments. The material nonlinearity is included in the FE analysis. The element type used for the numerical discretization in the 3D concrete parts is the C3D8R element from the ABAQUS library [20]. The T3D2 (3-noded quadratic 2-D) truss elements are used to generate textile reinforcements. Figure 9 also shows the meshing of the FE model for the concrete slab carbon textile. To achieve reliable results, the fine mesh was used in the pure bending zone. In these models, embedded textile reinforcement is assumed to be perfectly bonded to the concrete element, which implies infinite bond strength at the interface between the concrete and the reinforcement.

The material behaviour of single textile roving can be appropriately described with a brittle elastic isotropic material. The fiber strength controls the tensile failure of textile in the fiber direction. As seen from Figure 10-a, the textile material has captured elastic-brittle materials' response, and it showed no appreciable plastic deformation before failure. The stress is linear up to the tensile strength and does not exhibit yielding behaviour. After reaching the tensile strength, the stress drops sharply to zero, representing the textile's rupture. The input parameters were assigned according to the experimental data.

![Stress-strain relationship for the textile and concrete materials](image)
Table 3. The values of the parameters used in CDP model for concrete

<table>
<thead>
<tr>
<th>$E_c$ (MPa)</th>
<th>$f_{yc}$ (MPa)</th>
<th>$E_c'$ (MPa)</th>
<th>$v$</th>
<th>$K_c$</th>
<th>$\epsilon$</th>
<th>$\sigma_{yt}/\sigma_{yc}$</th>
<th>$\psi$</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>47.5</td>
<td>6.95</td>
<td>32600</td>
<td>0.2</td>
<td>2/3</td>
<td>0.1</td>
<td>1.16</td>
<td>$31^\circ$</td>
<td>1E-5</td>
</tr>
</tbody>
</table>

The Concrete Damaged Plasticity (CDP) model was adopted to model the inelastic stress-strain relation in the compressive and tensile regions (Figure 10-b). This model consists of the combination of non-associated multihardening plasticity and scalar damaged elasticity to describe the irreversible damage that occurs during the fracturing process [20]. For compressive behaviour, the uniaxial stress-strain curve of Eurocode 2 [21] is selected to determine yield stress and inelastic strain. For the tensile behaviour of concrete, the tensile stress was assumed to increase linearly to the strain until the concrete crack. After the cracking of the concrete, tensile stress decreases linearly to zero. In the CDP model, five parameters control the evolution and the shape of the yield surface and the flow potential. These parameters' default values are taken as a recommendation of the CDP model in ABAQUS user's manual [20]. These material properties that have been assigned in the CDP model are summarized in Table 3.

In total, five three-dimensional finite element models were developed. Three of them, denoted as P0L1-FEM, P0L2-FEM, and P0L3-FEM, were non-prestressed slabs with 1, 2, and 3 textile layers, respectively. Two models, namely P50L1-FEM and P70L1-FEM, were prestressed slabs. In the prestressed models, the function “predefined field” in ABAQUS was used to introduce prestressing in the textile reinforcement. The prescribing field stress variable values were defined as 1400 MPa and 2000 MPa, corresponding to 50% and 70% prestressed stress in textile reinforcement (Figure 11).

3.2. Verification of the FE Models

The load-deflection curves of all five models are plotted in Figure 12-a. In the case of 3 non-prestressed slabs, it is apparent that increasing the number of layers tends to enhance the overall capacity of the TRC slabs. At the same level of applied load, a high amount of textile reinforcement corresponds with a lower deflection. Therefore, the performance of the TRC slabs in the service limit state is increased. To verify the FE models, the numerical and experimental load-deflection curves are also compared with each other and illustrated in Figure 12-b, c, d. It can be seen that the cracking and ultimate loads obtained from the FE model correlate relatively well to those from the tests. A comparison between the cracking and ultimate loads is presented in Table 4. The average discrepancy in cracking and ultimate load values between experimental and FE predictions were found to vary from 2 to 13%.

As clearly seen in Figure 12-b, the comparison between FE results and experimental data for one textile layer slab shows a good correlation for whole curves. The P0L1-FEM slab behaved like an elastic material up to the first cracking strength. Then, the first cracks in the concrete appear exposed by a sudden variation of the load. The stabilization of the cracking follows this process and subsequently increasing tensile stress in textile reinforcement. The stiffness of the slab is reduced, and the textile reinforcement stress increases until the breaking point. The breaking of carbon textile in the P0L1-FEM model is also numerically depicted by the Von Mises stress distribution shown in Figure 13-a. As can be seen, at the applied load of 19.89 kN, the tensile stresses of the carbon layer in the pure flexural zone exceeded the textile's tensile strength. Then, with the increase of the applied displacement, the tensile stress of
textile at these positions dropped suddenly to zero, representing textile reinforcement's rupture. It is also noted that the compressive stress in the concrete slab was much smaller than the compressive strength. It shows the good agreement in failure mode to the experiment results.

### Table 4. Comparison of experimental and numerical results

<table>
<thead>
<tr>
<th>Set</th>
<th>Slab</th>
<th>Cracking load</th>
<th>Ultimate load</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Experiment</td>
<td>FEM</td>
<td>EXP</td>
<td>FEM</td>
<td>EXP</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$P_{cr}$ (kN)</td>
<td>$P_{cr,avg}$ (kN)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$P_u$ (kN)</td>
<td>$P_{u,avg}$ (kN)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Set 1: Non-Prestress</td>
<td>P0L1N1</td>
<td>5.5</td>
<td>5.32</td>
<td>5.62</td>
<td>1.06</td>
<td>21.4</td>
<td>5.62</td>
<td>1.06</td>
<td>21.4</td>
</tr>
<tr>
<td></td>
<td>P0L1N2</td>
<td>5.48</td>
<td>5.32</td>
<td>5.62</td>
<td>1.06</td>
<td>20.4</td>
<td>20.51</td>
<td>19.89</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>P0L1N3</td>
<td>4.97</td>
<td>6.02</td>
<td>6.43</td>
<td>1.07</td>
<td>25.38</td>
<td>25.36</td>
<td>28.65</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td>P0L2N1</td>
<td>6.10</td>
<td>6.02</td>
<td>6.43</td>
<td>1.07</td>
<td>25.38</td>
<td>25.36</td>
<td>28.65</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td>P0L2N3</td>
<td>5.60</td>
<td>6.22</td>
<td></td>
<td></td>
<td>24.99</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Set 2: Prestress</td>
<td>P0L1N1</td>
<td>9.89</td>
<td>6.10</td>
<td>6.02</td>
<td>14.35</td>
<td>1.48</td>
<td>22.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P0L1N2</td>
<td>9.15</td>
<td>6.37</td>
<td>6.23</td>
<td>6.78</td>
<td>1.09</td>
<td>22.11</td>
<td>21.91</td>
<td>22.28</td>
</tr>
<tr>
<td></td>
<td>P0L1N3</td>
<td>10.11</td>
<td>9.44</td>
<td>9.86</td>
<td>17.35</td>
<td>1.76</td>
<td>21.69</td>
<td>21.33</td>
<td>24.04</td>
</tr>
</tbody>
</table>

Figure 12. Comparison of numerical and experimental results of the Non-Prestressed slabs in Set 1
Moreover, the models of the slabs using 2 and 3 layers of textile reinforcement were analyzed. As expected, their behavior was very similar to the case of 1 layer of textile. The elastic phase is almost identical in all cases, as shown in Figure 12. The models with 2 and 3 textile layers had the same failure mode, the crushing of concrete slabs before reaching the textile’s rupture in tension. Figure 13-b shows that, at failure points of the POL2-FEM model, the textile stress is only 2108 MPa. Simultaneously, the compressive stresses in extreme concrete fibers were even higher than the compressive strength of concrete due to the confinement in local positions (under the applied load points). After this point, the stress distribution in concrete changed dramatically, representing the crushing in concrete. This leads to a gradual decrease of the load-deflection curves. The main differences between FEM models were tested specimens were the models’ stiffness after cracking and the ultimate load. As seen in Figure 12-c,d, both models, POL2-FEM, and POL3-FEM, were slightly stiffer than the actual slabs after cracking point.

Figure 14 contains a comparison between the load-deflection curves predicted by ABAQUS and the test results for prestressed slabs. The cracking load in P50L1-FEM and P70L1-FEM are respectively 48 and 76% higher than those of tested specimens. This could be explained by the reason that the prestressing force does not remain constant during the manufacturing process. The loss of stress in textile reinforcements was not well-controlled and also not implemented.
in ABAQUS models. However, the ultimate load in these models are marginally bigger than those at tested slabs, corresponding to a difference of 2% in P50L1-FEM. The large difference in stiffness of FE model also caused the much smaller deflection at the failure point. In the P50L1-FEM model, the textile was ruptured at displacement of 3.4 mm, while those in the experiments were was approximately 6 mm.

![Stresses in textile and concrete slab at the time of prestress releasing](image1)

![Stresses in textile and concrete slab before failure](image2)

**Figure 15.** Von Mises stress distribution in concrete and textile in prestressed model P50-FEM

Figure 15 represents the Von Mises stress distribution in concrete and textile in prestressed model P50-FEM. When releasing the prestress load, the stress in the longitudinal rovings was uniformly distributed approximately 1400 MPa. The stress in concrete was also uniformly dispersed in the whole slab. This can be explained by the bond between the concrete and textile reinforcements are assumed to be perfect (no-slip) in the FE analyses. The stresses in the bottom fibers of the concrete slab were transferred from compression to tension by increasing the applied loads. As can be seen in Figure 15-b, both two prestressed models were failed due to textile rupture. At this point, the tensile stresses of the carbon layer in the pure flexural zone exceeded the textile's tensile strength, while the compressive stress in concrete was still smaller than the compressive strength.

### 4. Conclusions

This paper presents the experimental and numerical results obtained for the flexural behaviour of carbon TRC slabs. Both the analytical and experimental works demonstrate that it is possible to use carbon textile to replace steel bars to reinforce and prestress the precast slabs. The influences of the number of textile layers and prestressing grades on the flexural performances are investigated using four-point bending tests. Following conclusions could be drawn from the current investigation:

For the non-prestressed slabs, increasing the number of textile layers has a significant effect on overall behaviour. With the increase in the number of textile layers, an improvement in the specimens' bearing capacity and a smaller reduction in the flexural stiffness of the cracked specimens were observed. However, due to the limitation of the concrete compression capacity limits, the textiles' tensile strength was not fully utilized. The specimen's failure mode changed from textile rupture in 1 layer specimens to concrete crushing failure in 2 and 3 layers specimens.

Prestressing force on textile contributes to the evident improvement on first-crack load, but only slightly influences the ultimate tensile strength of TRC. The average first-crack load of the prestressed specimens increased by 83.3% and 86.7%, respectively, compared with those of non-prestressed slabs. However, it should be noted that the effectiveness of prestressing force grades (i.e. 50% and 70%) in cracks resistance is not much different. The presence of prestressing force also improved both load and energy dissipation of TRC slabs at the limit of deflection in serviceability.

The proposed FE models could accurately predict the load-carrying capacities and load-deflection relationships for the non-prestressed slabs. For the prestressed slabs, there were significant differences in cracking load between numerical models and test specimens. This could be explained by the reason that the prestressing force does not remain constant during the manufacturing process. The loss of stress in textile reinforcements was not well-controlled and also not implemented in ABAQUS models.
5. Declarations

5.1. Author Contributions
The basic theme of the research was discussed and decided by D.Q.N. and H.C.N. The manuscript was written by H.C.N. The results, discussions and conclusion section was completed by D.Q.N. and H.C.N. All authors have read and agreed to the published version of the manuscript.

5.2. Data Availability Statement
The data presented in this study are available in article.

5.3. Funding
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5.4. Acknowledgements
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5.5. Conflicts of Interest
The authors declare no conflict of interest.

6. References


