



GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

BUILDING MATERIALS AND STRUCTURES

Volume 69

September 2026

ISSN 2217-8139 (Print)

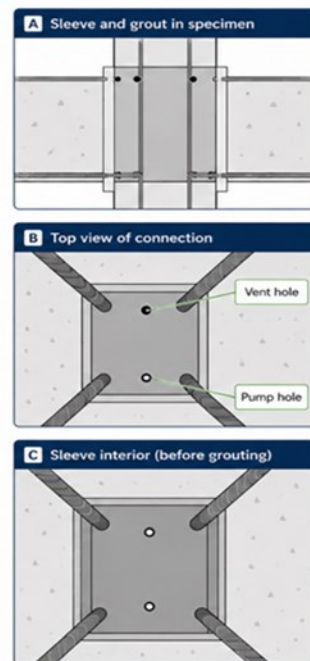
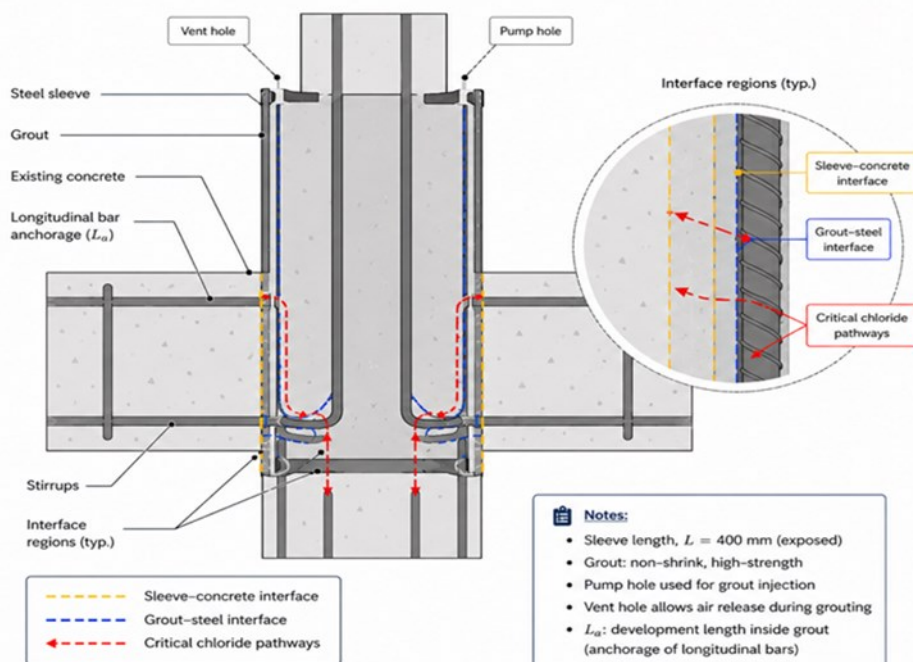
ISSN 2335-0229 (Online)

UDK: 06.055.2:62-

03+620.1+624.001.5(49
7.1)=861

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Society for Materials and Structures Testing of Serbia
University of Belgrade Faculty of Civil Engineering
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Publishers	Society for Materials and Structures Testing of Serbia, Belgrade, Serbia, veb sajt: www.dimk.rs University of Belgrade Faculty of Civil Engineering, Belgrade, Serbia, www.grf.bg.ac.rs Association of Structural Engineers of Serbia, Belgrade, Serbia, dgks.grf.bg.ac.rs
Print	Razvojno istraživački centar grafičkog inženjerstva, Belgrade, Serbia
Edition	quarterly
Peer reviewed journal	
Journal homepage	www.buildingmaterialsstructures.com
Cover	<i>Schematic of the grouted sleeve connection detail used in the prefabricated beam-column joint, showing the steel sleeve, grout, existing concrete, longitudinal bar anchorage (L_a), stirrups, pump hole, vent hole and the critical chloride pathways along the sleeve-concrete and grout-steel interfaces</i> by Md Mahmudul Hasan and Pradyut Anand
Financial support	Ministry of Education, Science and Technological Development of Republic of Serbia University of Belgrade Faculty of Civil Engineering Faculty of Technical Sciences, University of Novi Sad, Department of Civil Engineering Serbian Chamber of Engineers

CIP - Каталогизacija y publikaciji
Народна библиотека Србије, Београд

620.1

GRAĐEVINSKI materijali i konstrukcije = Building materials and structures / editor-in-chief Snežana Marinković
. - God. 54, br. 3 (2011)- . - Belgrade : Society for Materials and Structures Testing of Serbia : University of Belgrade, Faculty of Civil Engineering : Association of Structural Engineers of Serbia, 2011- (Belgrade : Razvojno istraživački centar grafičkog inženjerstva). - 30 cm

Tromesečno. - Je nastavak: Materijali i konstrukcije
= ISSN 0543-0798. - Drugo izdanje na drugom medijumu:
Građevinski materijali i konstrukcije (Online) = ISSN 2335-0229
ISSN 2217-8139 = Građevinski materijali i konstrukcije
COBISS.SR-ID 188695820



Original scientific paper

Interface-controlled chloride corrosion in grouted sleeve joints of prefabricated RC elements: Performance enhancement using duplex stainless steel (Grade 2205)Md Mahmudul Hasan¹⁾, Pradyut Anand^{*1,2)} ¹⁾ Department of Civil Engineering, School of Engineering & Technology, Noida International University, Greater Noida, Uttar Pradesh 201312, India²⁾ Department of Civil Engineering, Graphic Era (Deemed to be University), Dehradun 248002, Uttarakhand, India*Article history*

Received: 02 May 2026

Received in revised form:

23 May 2026

Accepted: 05 June 2026

Available online: 06 July 2026

*Keywords*prefabricated concrete,
corrosion behaviour,
chloride ingress,
grouted sleeve connection,
interfacial transition zone (ITZ),
duplex stainless steel (Grade 2205),
durability performance**ABSTRACT**

Prefabricated reinforced concrete (RC) systems are efficient in terms of construction, but their durability is highly dependent on the behaviour of joint regions with several material interfaces. This research examines the chloride-induced corrosion performance of prefabricated beam–column joints connected by grouted sleeves, as compared with traditional cast-in-situ counterparts. The study employed an experimental–analytical method with wetting-drying cycles in a 3.5% NaCl solution to represent severe environmental exposure. The findings show that prefabricated joints demonstrate faster corrosion, with corrosion initiation 40–60% earlier than in cast-in-situ joints. The rate of corrosion was about 2.8 times faster, and the chloride diffusion coefficient increased by 2.5 times, confirming increased transport via the interfacial regions. Damage was concentrated at the sleeve interfaces, with a Damage Localisation Index (DLI) of about 0.62, and led to wider cracks than observed in traditional structures (more than twice the width). This study shows that the corrosion behaviour of prefabricated joints is largely controlled by the interface rather than the bulk properties. Targeted application of duplex stainless steel (Grade 2205) in key areas was effective in delaying the onset of corrosion and in diminishing its impact, showing the benefit of using this material in critical areas. The research provides evidence of interface-controlled corrosion and highlights the need for interface engineering and material design to enhance the sustainability of prefabricated concrete structures.

1 Introduction

Prefabricated reinforced concrete (RC) systems are gaining momentum in contemporary construction for their benefits in speed, quality and sustainability [1–4]. Such systems are extensively used in infrastructure works and buildings where time is of the essence. But their structural integrity relies on the performance of their joints or connections, which are the main load-bearing zones and often the most vulnerable parts of prefabricated structures.

In contrast to the cast-in-situ approach, prefabricated systems include multiple interfaces between various materials, such as concrete–grout, grout–steel, and sleeve–concrete [5–7]. These boundaries introduce heterogeneity and are often more porous, less bonded and microscopically discontinuous. The higher porosity, poorer bonding and existence of microvoids and discontinuities make the interface more prone to damage under aggressive environmental conditions [8–11]. These characteristics i.e. higher porosity, poorer bonding and existence of microvoids

and discontinuities are usually associated with variations in material characteristics, surface properties and construction processes, such as voids, air pockets, and poor surface preparation. The interfaces are thus more susceptible to damage, especially when subjected to hostile environments.

Chloride-induced reinforcement corrosion is the most significant durability issue that impacts the performance of reinforced concrete. In traditional systems, chloride penetration is usually assumed to be a diffusion process through a dense and homogeneous concrete matrix [12–14]. Yet, in prefabricated joints, this process is quite different. Chloride ions are more likely to travel through the concrete via pathways of lower resistance (such as interfaces), resulting in accelerated accumulation at the reinforcing steel.

This preferential transport is exacerbated by uneven moisture, oxygen and ionic concentrations within the joint. These differences promote the establishment of local anodic and cathodic sites, leading to pitting. These forms of corrosion are especially aggressive at sleeve ends, grout imperfections, and construction joints such as pump and vent

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holes, where corrosion initiation and propagation is more likely. This can result in considerable loss of steel cross-section, bond deterioration and a decrease in strength over time.

Such effects are particularly prevalent in conventional reinforcement (Fe-415 and Fe-500) as it tends to lose its passive layer in high-chloride environments. This issue is more prevalent in precast systems, in which the corrosion performance is highly dependent on the interface. To address this issue, duplex stainless steel (Grade 2205) has gained traction as a viable alternative material, thanks to its high strength and excellent resistance to chloride attack. Its capacity to sustain a stable passive layer and resist pitting corrosion make it an excellent candidate for application in critical zones where corrosion is likely to occur.

Although prefabricated construction is gaining momentum, most of the previous studies on reinforced concrete corrosion have been related to cast-in-situ construction and have assumed uniform material properties. This means that traditional models are primarily based on bulk diffusion and do not sufficiently account for the effect of interfaces, which are predominant in prefabricated joints. The role of interfaces, defects and discontinuities in providing local transport pathways to corrosion initiation and growth is poorly understood.

Furthermore, although corrosion-resistant materials have been investigated for general use in reinforced concrete, they have received little consideration for use in the critical interfacial areas of prefabricated joints. Specifically, the performance of duplex stainless steel in situations where corrosion is controlled by interfacial properties has not been rigorously investigated. This underscores a need to better understand the interplay between material choice and interface characteristics in prefabricated systems.

To overcome these limitations, this research examines the corrosion of reinforcement in prefabricated beam-column joints with grouted sleeve connections exposed to chlorides, with particular emphasis on the effect of interfaces. An experimental-analytical framework was used to investigate corrosion initiation, propagation and damage localisation, and to compare prefabricated and cast-in-situ systems. Moreover, the study examines the suitability of duplex stainless-steel Grade 2205, in accordance with ASTM A240/A240M, as a targeted reinforcement approach in critical regions.

Key findings of this study include:

- Developing an interface-controlled corrosion process, demonstrating that the transport of chlorides and corrosion initiation are largely controlled by the interface rather than the concrete matrix.
- Development of a Damage Localisation Index (DLI) to measure how much and how concentrated the corrosion damage is in prefabricated joints.
- Offering experimental evidence under accelerated chloride attack, showing the difference in time to corrosion initiation, corrosion rate and damage evolution between prefabricated and cast-in-situ systems.
- Showing the potential benefits of using duplex stainless steel (Grade 2205) as a local mitigation measure to limit localised corrosion at critical locations.

1.1 Chloride-induced corrosion in prefabricated joints

Corrosion of RC structures is a well-known degradation process, especially in marine and de-icing salt environments. The conventional cast-in-situ systems generally have a

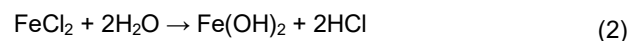
relatively uniform and dense concrete matrix through which the diffusing chloride is usually moving. In the case of prefabricated RC structures, however, the chloride ingress mechanism has a strong dependency on the material interfaces and construction discontinuities, and its corrosion behaviour is highly interface-sensitive.

Prefabricated beam-column connections with grouted sleeve joints are composed of a number of distinct regions, including concrete-grout interfaces, grout-steel interfaces and sleeve-concrete interfaces. These regions are more porous and susceptible to microcracking and a lack of bonding because of incompatibility in material properties, shrinkage, surface preparation, and construction errors such as air voids or poor compaction of grout. As a result, these zones serve as potential channels for the penetration of chloride ions, water and oxygen, giving rise to a highly non-homogeneous and localised diffusion process compared to monolithic concrete.

Once the chloride ions penetrate through these susceptible zones and reach the reinforcing steel, they disrupt the alkaline passive oxide layer normally formed on the steel surface. The reaction between ferrous ions and the chloride ions results in the formation of soluble iron chloride:



One of the distinctive features of chloride-induced corrosion is that it is self-catalysing. The iron chloride formed reacts with water to form ferrous hydroxide and hydrochloric acid:



The hydrochloric acid further lowers the pH, thereby enhancing the depassivation and dissolution of steel. This regeneration process allows the corrosion process to continue and accelerate, even in the absence of further chloride ingress, making it highly corrosive in small and susceptible interfacial areas.

In prefabricated joints, higher chloride concentrations are typically found in the vicinity of interfaces (e.g., sleeve ends, grout-concrete interface), and construction joints (e.g., pump hole and vent hole). Such microclimatic conditions enable the formation of electrochemical gradients caused by differences in moisture, oxygen and ion concentrations. This effectively creates corrosion cells that result in pitting as opposed to uniform corrosion. This type of damage is particularly insidious because of the rapid loss of cross-section of the reinforcement.

The sleeve (steel or polymer)-cementitious material interface is of particular importance. Lack of bonding and thermal or shrinkage compatibility can result in micro-gaps, which can act as both pathways for chlorides and sites for cracking and corrosion to initiate. Likewise, variations in the quality of bonding between grout and steel in sleeves also enhance corrosion.

Previous experimental studies have indicated that localised corrosion is more likely to occur at the vicinity of interfaces between sleeves, grout discontinuities and construction defects than in relatively homogenous concrete areas [3]. Wang et al. [14] found that accelerated corrosion occurred near the interfaces of grouting in the case of chloride exposure, and Li and Luo [4] showed that the existence of defects at the interfaces of prefabricated joints greatly promoted chloride transport and localised corrosion. In summary, corrosion in prefabricated joints is determined not just by the intrinsic properties of materials, but more importantly, by the nature of interfaces. The existence of preferential transport paths, microstructural imperfections

and electrochemical heterogeneities promotes the initiation and development of corrosion. Consequently, upgrading interfacial integrity by better matching material properties, surface treatment, and construction procedures is a key strategy to reduce the durability risks and enable long-term reliable performance of prefabricated reinforced concrete structures.

1.2 Aim

This research will explore interface-controlled corrosion in prefabricated RC beam-to-column joints exposed to chlorides and assess targeted strategies using duplex stainless steel.

2 Literature review

Reinforcement deterioration in RC structures is affected significantly by exposure to chloride surroundings. There has been considerable research on chloride ingress and the initiation of corrosion, where concrete is assumed to be a relatively homogeneous material. Traditional approaches based on Fick's Second Law have been commonly applied to model chloride ingress and service life [1], [2]. These methods have been largely successful in modelling cast-in-situ concrete but may miss the complexities associated with interfaces in prefabricated concrete.

Recent research has shown that the durability of prefabricated reinforced concrete structures is different from that of cast-in-situ concrete structures due to the existence of multiple interfaces. Bao et al. [3] and Li and Luo [4] have demonstrated that grouted sleeve connections exhibit higher porosity and reduced interfacial bonding between grout and concrete, which can lead to increased chloride penetration. Similarly, Liao et al. [5] showed through microstructural investigations that the interfacial transition zone (ITZ) of prefabricated connections often has microvoids and defects that provide transport pathways for aggressive ions.

The significance of these interfacial defects in controlling transport has been highlighted by studies of permeability and diffusivity. For example, Li et al. [6] found that chloride ingress in prefabricated elements is highly localised, with much higher concentrations at the interfaces than in the bulk concrete. This observation complicates the notion of uniform diffusion and indicates that chloride transport in these systems is strongly affected by the local transport pathways, rather than the overall material properties.

Besides transport, some researchers have considered the electrochemical aspects of corrosion in non-uniform concrete systems. Potgieter and Marjanovic [7] reported that differences in moisture and oxygen content can result in local anodic and cathodic areas, leading to pitting. This is especially important in prefabricated joints, where there are heterogeneities that promote local electrochemical processes. Li and Luo [4] reported that joint and defect regions of reinforcement are subject to more intense and localised corrosion than the surrounding reinforcement in uniform concrete.

Various methods to protect materials from corrosion have been studied, such as protective coatings, corrosion inhibitors and stainless-steel reinforcement. Of these, Grade 2205 duplex stainless steel has the highest corrosion resistance and mechanical properties. However, due to its relatively high initial cost compared with traditional carbon steel, it is more economical to apply it in specific application areas, especially in critical interfacial regions. Research by Pérez-Quiroz et al. [8] and Duarte et al. [9] showed that

duplex stainless steel retains its passivity at elevated chloride levels, greatly improving its lifespan. But most of these studies were conducted under uniform exposure and do not specifically investigate behaviour in prefabricated joints that are prone to localised corrosion.

However, there remains a significant gap in knowledge about corrosion in prefabricated reinforced concrete. Most of these studies are based on models for uniform materials and overlook the role of interfaces in chloride transport and initiation of corrosion. Additionally, the spatial variability of corrosion damage — especially in vulnerable locations such as sleeve boundaries and grout defects — has yet to be quantified in detail. Although laboratory experiments indicate the importance of interfacial zones, a holistic approach to relate microstructural features, transport processes and corrosion mechanisms remains to be established.

Additionally, the idea of mitigation (using corrosion-resistant materials) in high-risk areas has rarely been considered. Specifically, the role of duplex stainless steel in vulnerable interfaces of prefabricated joints under accelerated chloride attack has yet to be explored [10], [11].

Hence, there is a need for a more realistic and mechanistic perspective on corrosion in prefabricated structures. The current study seeks to overcome these limitations by creating a mechanistic view of interface-controlled corrosion and experimentally testing this view in prefabricated beam-column connections with grouted sleeves. Previous studies do not consider the degree of corrosion localisation and do not explicitly account for interface-controlled transport, both of which are treated in the present study.

2.1 Objective

- Quantify corrosion initiation and propagation in prefabricated vs cast-in-situ systems.
- Characterise chloride transport at interfaces.
- Create a Damage Localisation Index (DLI) for corrosion spread.
- Evaluate the effectiveness of duplex stainless steel in key areas.

2.2 Novelty

- Establishes an interface-controlled mechanism of corrosion, instead of the traditional uniform diffusion.
- Develops a Damage Localisation Index (DLI) to measure non-uniform corrosion.
- Furnishes experimental evidence of enhanced kinetics in prefabricated joints.
- Shows effective localised mitigation by using duplex stainless steel.

3 Methodology

3.1 Overview

Localised corrosion behaviour was evaluated in sleeve-connected joints, with the help of experimental observation and analytical interpretation. For a systematic comparison, three types of specimens were examined: (i) cast-in-situ specimens (C), (ii) prefabricated specimens with grouted sleeve connections (PC), and (iii) modified prefabricated specimens with conventional reinforcement but using duplex stainless steel (Grade 2205) in critical locations (PC-DSS).

Geometric consistency was maintained for all specimens in terms of geometry and reinforcement arrangement, to exclude any effects on the corrosion behaviour. The use of duplex stainless steel was focused on critical areas, specifically the stirrups placed in close proximity to the sleeve–grout–concrete interface, to investigate its role as a localised anti-corrosion measure.

The details of the grouted sleeve connection and critical interfacial regions are shown in Fig. 1.

3.2 Materials

In this study, normal-grade concrete of grade M30 was used for the structural elements and high-performance cementitious grout was used in the sleeve connections. Standard deformed (Fe-500) steel bars were used for the longitudinal reinforcement and stirrups in the C and PC specimens, while in the PC-DSS specimens the longitudinal bars and the stirrups located within the joint zone (one stirrup spacing on either side of the sleeve – the highlighted DSS region in Fig. 2) were replaced with duplex stainless steel (Grade 2205, ASTM A240/A240M) bars as a corrosion-resistant material. Concrete compressive strength, steel yield strength and grout flowability were measured using standard laboratory tests. Special attention was paid to the compatibility between grout, steel, and concrete, as this is crucial for the performance of the interface and the durability of the entire system.

3.3 Exposure conditions and corrosion simulation

To simulate harsh environmental exposure, all specimens were exposed to alternating wet–dry cycles in a 3.5% solution of sodium chloride (NaCl) — a typical representation of marine or chloride-rich environments.

The exposure programme involved repetitive cycles of wetting and drying to hasten chloride ingress and induce corrosion in a laboratory environment over a shorter period. This approach allows the acceleration of field conditions within the timespan of the laboratory work and enables the investigation of the initiation and propagation of corrosion. The geometry, reinforcement details, exposure conditions and corrosion simulation set-up for the three specimen categories (C, PC, PC-DSS) are illustrated in Fig. 2.

3.4 Corrosion assessment techniques

The corrosion performance was assessed through electrochemical tests, visual observations, and post-exposure investigations. Periodic half-cell potential measurements were taken to detect corrosion activity and to determine when active corrosion began.

Visual inspection was also conducted to identify signs of damage such as rust discolouration, cracking and concrete spalling. The width of cracks was measured digitally to assess deterioration.

After exposure, the steel bars were removed from the specimens to measure the steel mass loss, and to evaluate the distribution of corrosion along the bar length. Furthermore, non-destructive imaging techniques such as X-ray computed tomography (CT) were used to assess internal damage, including thickness of the corrosion layer, voids and flaws at the interfaces.

The half-cell potential tests were performed in a manner consistent with ASTM C876, using a Cu/CuSO₄ reference electrode. The measurements were taken at predefined grid points in the beam–column joint area, with particular focus on the sleeve interfaces and the pump/vent hole positions where localised corrosion was predicted. The half-cell potential measurement arrangement and predefined grid locations are illustrated in Fig. 3.

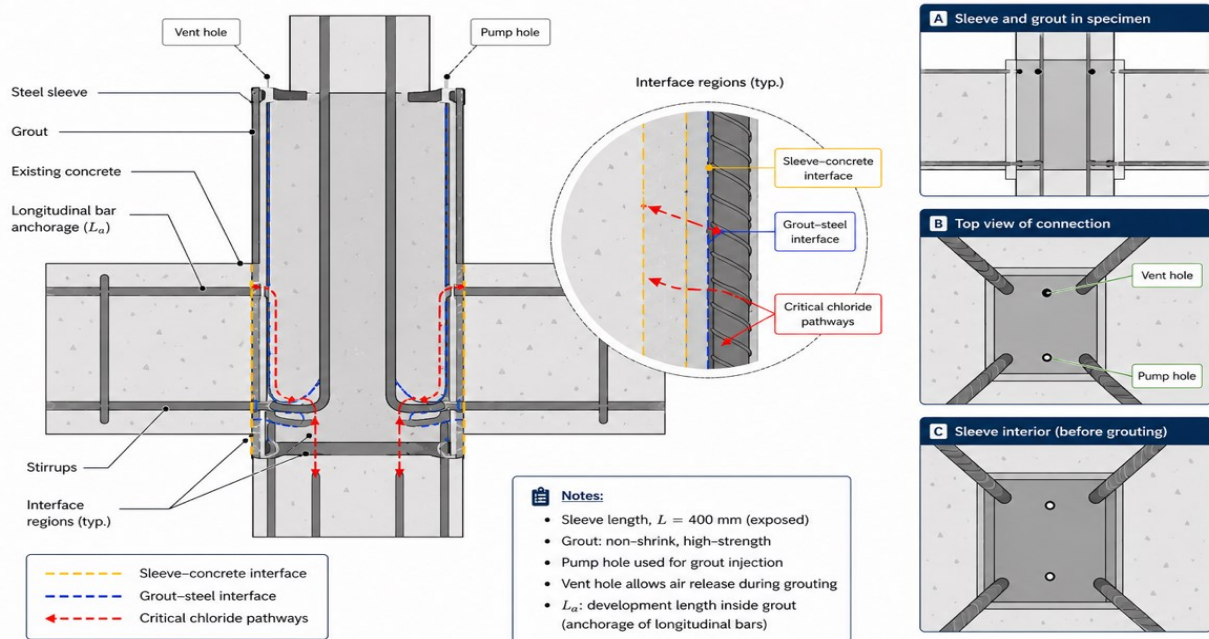


Figure 1. Schematic of the grouted sleeve connection detail used in the prefabricated beam–column joint, showing the steel sleeve, grout, existing concrete, longitudinal bar anchorage (L_a), stirrups, pump hole, vent hole and the critical chloride pathways along the sleeve–concrete and grout–steel interfaces

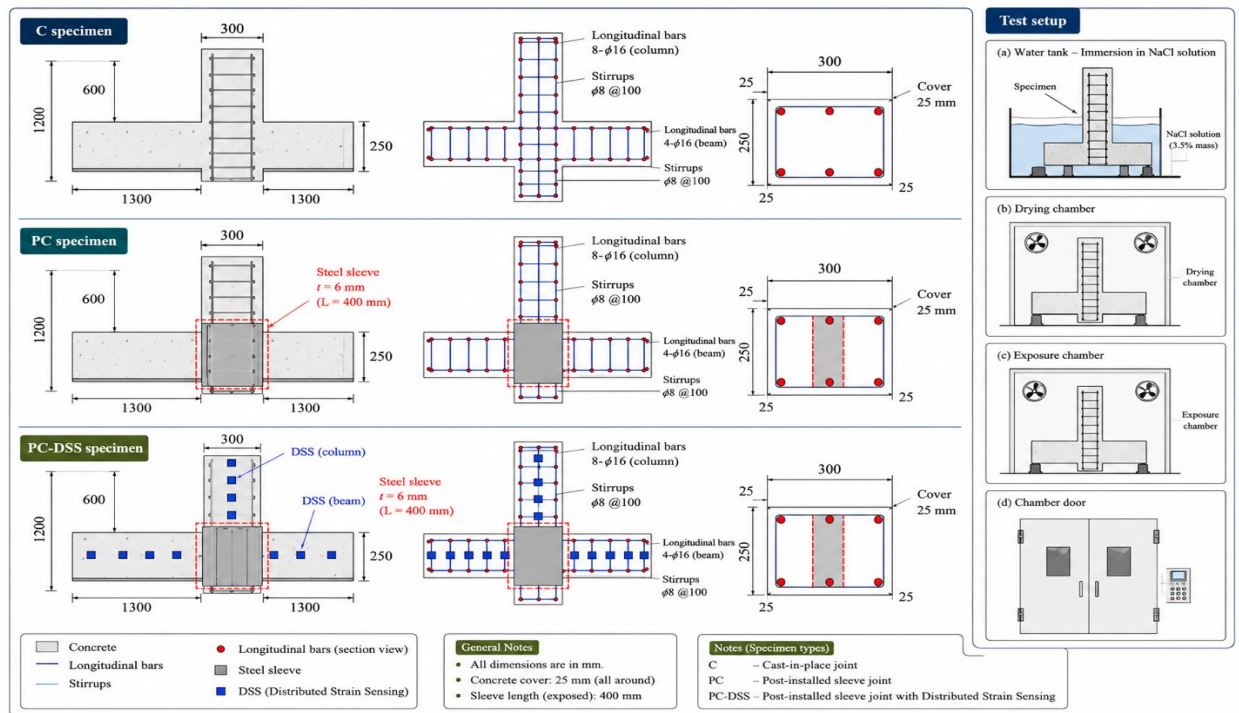


Figure 2. Geometry, reinforcement details and experimental set-up for the three specimen categories: (top) cast-in-situ (C), (middle) prefabricated with grouted sleeve (PC), and (bottom) prefabricated with duplex stainless-steel reinforcement in critical regions (PC-DSS). The right-hand panels show the test set-up: (a) NaCl immersion tank, (b) drying chamber, (c) exposure chamber, and (d) chamber door. All dimensions are in mm. Concrete cover: 25 mm; sleeve length (exposed): 400 mm. Red dashed regions indicate the grouted sleeve and the DSS placement zones

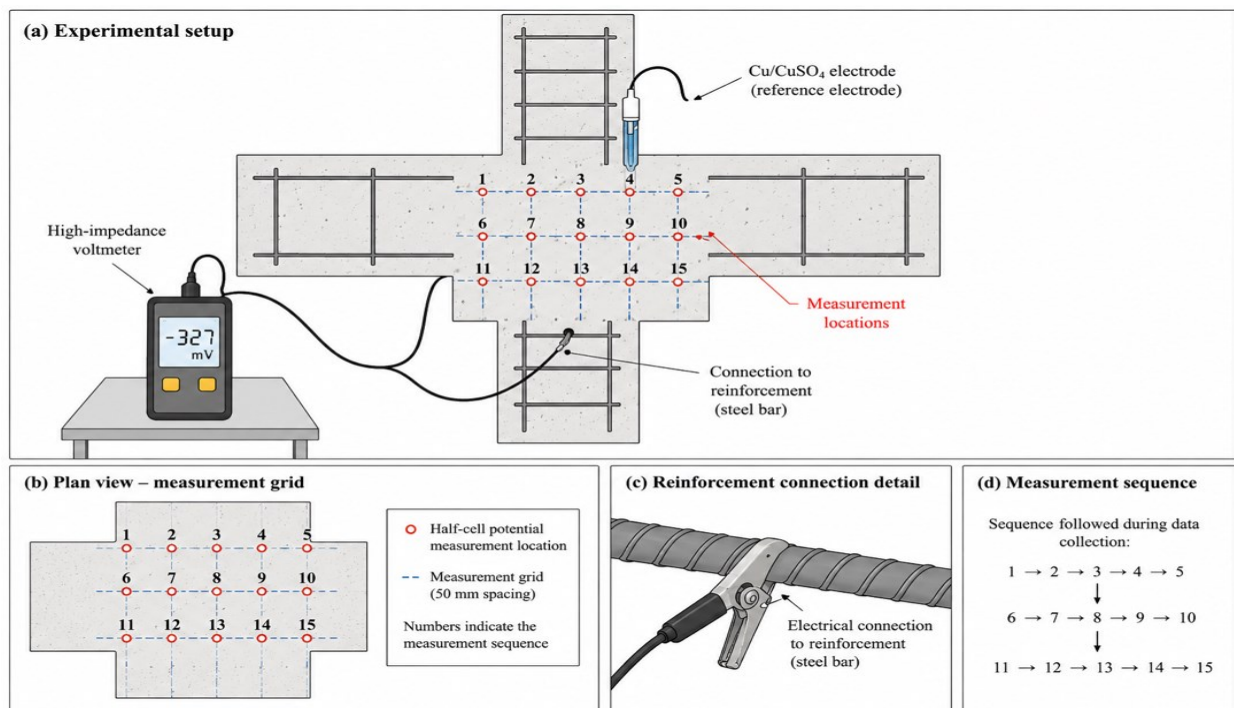


Figure 3. Half-cell potential measurement set-up. (a) Experimental set-up showing the high-impedance voltmeter, the Cu/CuSO₄ reference electrode and the connection to the reinforcement; (b) plan view of the measurement grid with 15 measurement locations spaced at 50 mm; (c) detail of the electrical connection to the steel bar; and (d) sequence followed during data collection (1 → 2 → ... → 15)

The corrosion rates were measured by linear polarisation resistance (LPR) and calculated as the average of three readings per specimen. The widest crack on each specimen was selected, and the crack width was measured using a digital crack microscope with a resolution of 0.01 mm.

The mass loss of the steel was measured after exposure by removing the reinforcement, cleaning the corrosion products following ASTM G1, and calculating the percentage of mass loss relative to the original bar mass.

The chloride diffusion coefficients were determined by inverse fitting of experimental chloride concentration profiles using the solution of Fick's second law. After 90 wetting–drying cycles (equivalent to 120 days), the chloride concentration was measured in powder samples at various depths from the exposed surface using titration analysis.

3.5 Interfacial characterisation

Since interfaces play a vital role in prefabricated structures, the interfacial zones — the sleeve–concrete interface and the grout–steel interface — were thoroughly investigated.

Microstructural examination was conducted to detect the presence of voids, micro-gaps and differences in bonding. These interfacial features were then linked with the level of corrosion to examine the correlation between interfacial properties and corrosion behaviour.

3.6 Analytical modelling of corrosion behaviour

A diffusion-based analytical model based on Fick's second law was developed to estimate chloride penetration and corrosion initiation at critical interface regions. The effective diffusion coefficient (D) for prefabricated specimens was approximately 2.5× higher than in cast-in-situ specimens, indicating a higher rate of chloride ingress at the

interface zones. The diffusion of chloride ions in the concrete matrix and regions was described by Fick's second law:

$$C(x, t) = C_s \cdot \left[1 - \operatorname{erf} \left(\frac{x}{(2\sqrt{D \cdot t})} \right) \right] \quad (3)$$

where $C(x, t)$ is the chloride concentration at depth x and time t , C_s is the surface chloride concentration, D is the apparent diffusion coefficient, and erf is the error function. The diffusion coefficient was determined by inverse analysis of the experimental data. An alternative formulation accounting for interfacial effects was also considered to capture the higher permeability in prefabricated joints.

3.7 Comparative Evaluation

A comparative study was undertaken to assess the corrosion behaviour of the three specimen categories. The parameters compared include time to corrosion initiation, corrosion rate, steel mass loss, crack-width evolution and Damage Localisation Index (DLI). This comparative approach enables the identification of key differences between cast-in-place and prefabricated construction systems and the role of duplex stainless steel in mitigating corrosion in joint zones.

3.8 Experimental design and statistical analysis

Five independent specimens ($N = 5$) were used for each category, with three replicate measurements per specimen. The corrosion-related parameters (corrosion rate, mass loss, crack width, and diffusion coefficient) are presented as mean $\pm$ standard deviation (SD), where SD is the variability among independent specimens. Triplicate measurements were used to reduce measurement uncertainty. The statistical significance of the results was determined by a two-tailed t -test, with $\alpha = 0.05$.

The experimental design is summarised in Table 1.

Table 1. Experimental design summary

Parameter	Description
Specimen Categories	C (Cast-in-situ), PC (Prefabricated), PC-DSS (Prefabricated with Duplex Stainless Steel)
Number of Specimens per Category (N)	5
Replicates per Specimen	3
Total Measurements per Category	15
Exposure Condition	Cyclic wetting–drying in 3.5% NaCl solution
Measured Parameters	Half-cell potential, corrosion rate, steel mass loss, crack width, diffusion coefficient
Data Reporting	Mean $\pm$ Standard Deviation (SD)
Error Bars	Represent SD across specimens
Statistical Test	Two-tailed t -test
Significance Level	$\alpha = 0.05$

Before the start of cyclic exposure, all specimens were visually documented to record their pristine surface condition (uniform light-grey concrete, no surface staining, no visible cracks, exposed sleeve free of corrosion). The corresponding post-exposure photographs of the extracted reinforcement are presented in Fig. 5 and provide the before/after comparison.

4 Results and discussion

This chapter discusses the results of the experimental and analytical work conducted and their interpretation. The section provides insights into the corrosion processes in prefabricated joints against cast-in-situ systems, and the role of the interface and the materials used on the durability of the system.

4.1 Corrosion initiation and half-cell potential

The variation of half-cell potential (E_{corr}) with exposure cycles is shown in Fig. 4. A steady decline in potential is observed in all samples, reflecting the progression of corrosion.

The PC specimens showed a much more rapid decrease in potential towards negative values than the C specimens.

The potential value of -350 mV (corresponding to a high risk of active corrosion) was reached at around 35 cycles for the PC specimens, while the C specimens reached this threshold at around 62 cycles. The PC-DSS specimens did not reach the -350 mV threshold within the test duration (>70 cycles).

The corresponding numerical results are summarised in Table 2.

As shown in Table 2, prefabricated specimens reached the critical corrosion threshold significantly earlier than cast-in-situ specimens. This premature corrosion in prefabricated joints is due to the presence of defects and increased permeability, which allow for quicker chloride ingress. On the other hand, the PC-DSS specimens exhibited a delayed potential shift, suggesting better resistance to the initiation of corrosion, which can be attributed to the higher passivity of duplex stainless steel.

4.2 Steel mass loss and corrosion distribution

Photographs of the reinforcement bars extracted after exposure testing are presented in Fig. 5. The pristine DSS bars shown in the right-hand column also serve as the before-testing reference for the corroded carbon-steel stirrups and longitudinal bars on the left and centre columns.

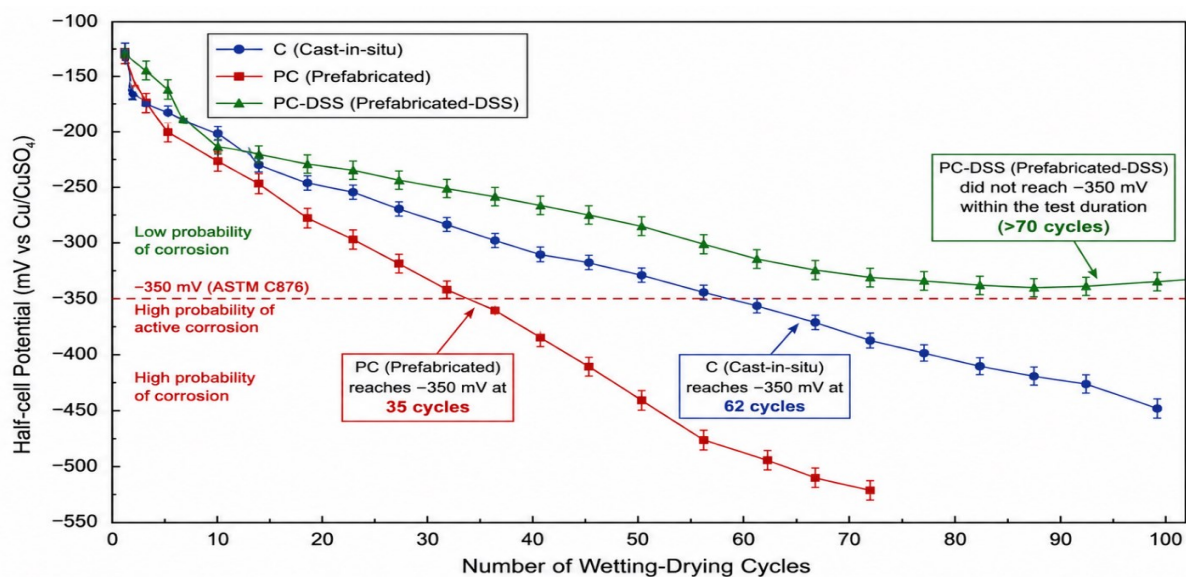


Figure 4. Variation of half-cell potential (E_{corr}) with wetting–drying exposure cycles for cast-in-situ (C), prefabricated (PC), and prefabricated with duplex stainless steel (PC-DSS) specimens under 3.5% NaCl exposure. The dashed line at -350 mV (vs. Cu/CuSO_4) represents the threshold for high probability of active corrosion according to ASTM C876. Corrosion initiation occurred after approximately 35 cycles for PC specimens and 62 cycles for C specimens, whereas PC-DSS specimens did not reach the corrosion threshold within the experimental duration (>70 cycles). Error bars represent standard deviation ($N=5$)

Table 2. Corrosion initiation and half-cell potential results

Specimen Type	Cycles to reach -350 mV	Corrosion Initiation (years, equivalent)	Observation
C	62	4.6	Delayed corrosion initiation
PC	35	1.8	Early corrosion initiation
PC-DSS	>70 (approx.)	Significantly delayed	Improved corrosion resistance

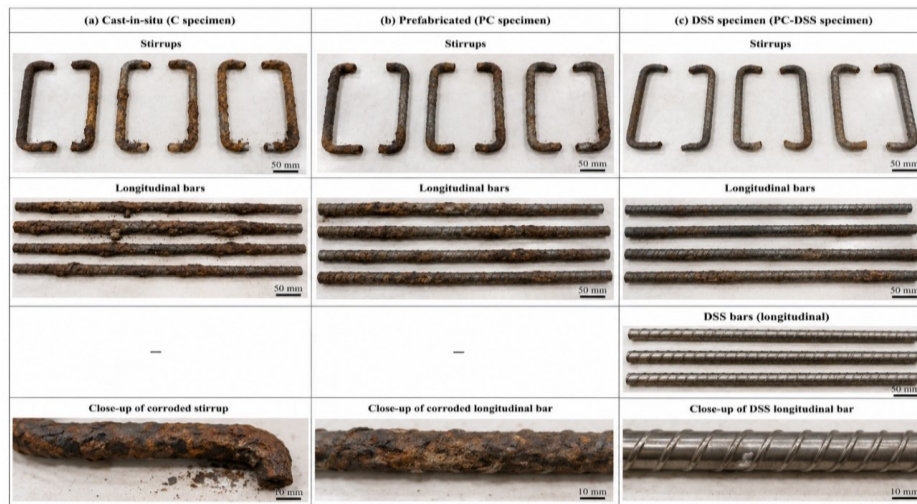


Figure 5. Extracted reinforcement after testing. (a) Cast-in-situ (C) specimen — stirrups, longitudinal bars and close-up of corroded stirrup. (b) Prefabricated (PC) specimen — stirrups, longitudinal bars and close-up of corroded longitudinal bar near the sleeve. (c) DSS specimen (PC-DSS) — stirrups, longitudinal carbon-steel bars (outside the joint zone) and pristine duplex stainless-steel (Grade 2205) longitudinal bars used inside the joint zone. The pristine appearance of the DSS bars after the same exposure programme contrasts with the heavy localised pitting observed on the carbon-steel stirrups and bars

Table 3. Steel mass loss and corrosion distribution

Specimen Type	Stirrups Mass Loss (%)	Longitudinal Bars (%)	Key Observation
C	2.8	2.1	Uniform corrosion
PC	7.6	2.5	Localised corrosion ($\approx 2.7\times$ higher in stirrups)
PC-DSS	Negligible (<0.5)	Negligible (<0.5)	Corrosion effectively mitigated

The steel mass loss and corrosion-distribution results for the different specimen categories are summarised in Table 3 and the time-dependent cumulative mass loss is illustrated in Fig. 6. As shown in Table 3, stirrups in prefabricated joints suffered approximately 2.7 times more mass loss than those in cast-in-situ specimens, while the difference for longitudinal bars was comparatively small.

This indicates that the corrosion in the prefabricated joints is highly concentrated. The localised damage in the vicinity of the sleeve junction and pump/vent holes shows the influence of discontinuities in controlling corrosion. For PC-DSS specimens, mass loss was minimal, demonstrating the excellent corrosion resistance of DSS in a chloride environment.

4.3 Corrosion rate analysis

The corrosion rates extracted by linear polarisation resistance confirm the trends discussed above. The corrosion rate of the prefabricated specimens was approximately 0.145 mm/year, almost 2.8 times that of the cast-in-situ specimens (0.052 mm/year). The numerical values are reported in Table 4 along with the chloride diffusion coefficients.

This shows that the interface permeability controls chloride ingress in prefabricated specimens. The PC-DSS specimens exhibited a much lower corrosion rate, suggesting that the localised application of corrosion-resistant reinforcement was effective in reducing deterioration in the system. This indicates that prefabricated joints are likely to have a significantly shorter service life unless properly mitigated.

Table 4. Corrosion rate and chloride diffusion characteristics.

Specimen Type	Corrosion Rate (mm/year)	Diffusion Coefficient (m^2/s)	Relative Increase
C	0.052	0.9×10^{-12}	Baseline
PC	0.145	2.3×10^{-12}	$\sim 2.8\times$ (rate), $\sim 2.5\times$ (diffusion)
PC-DSS	Very low (<0.01)	Similar to PC	Delayed corrosion response

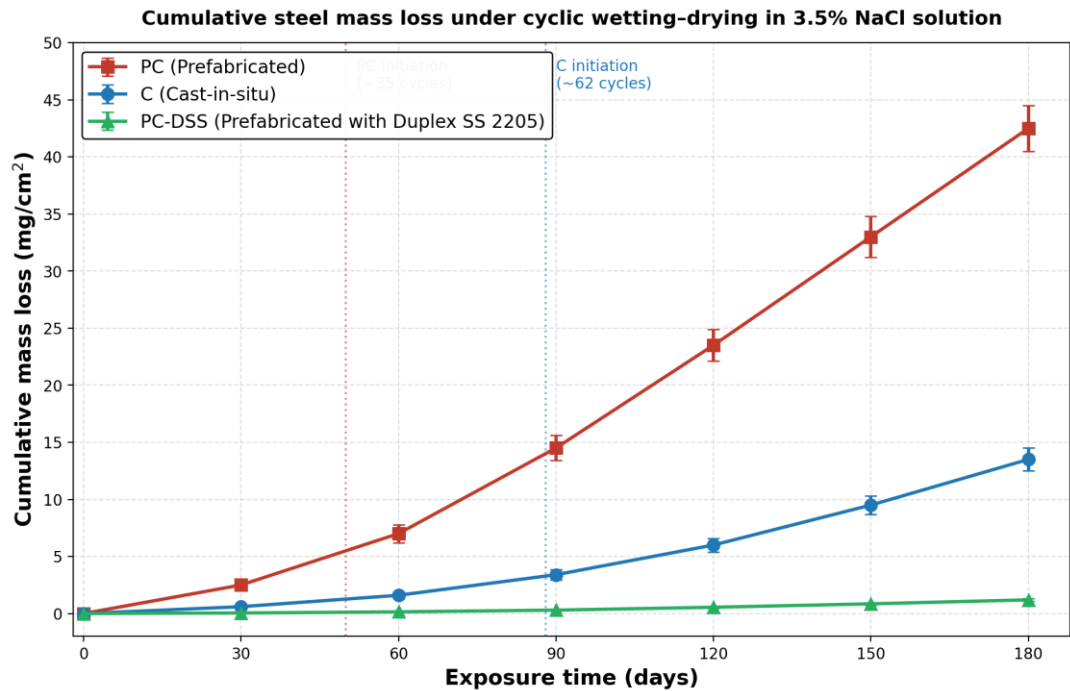


Figure 6. Cumulative steel mass loss (mg/cm^2) as a function of exposure time (days) for C, PC and PC-DSS specimens under cyclic wetting–drying in a 3.5% NaCl solution. The PC specimens display a sharp acceleration after the corrosion-initiation threshold (~ 35 cycles ≈ 50 days). C specimens accumulate damage more gradually after their later initiation (~ 62 cycles ≈ 88 days), and PC-DSS specimens remain almost unaffected over the entire 180-day programme. Error bars: $\pm \text{SD}$, $N = 5$

Note: Fig. 6 presents the cumulative mass loss in mg/cm^2 of exposed steel surface as a function of exposure time, while Table 3 reports the percentage mass loss at the end of the exposure programme (after specimen extraction and ASTM G1 cleaning). The two are complementary expressions of the corrosion damage and use different units; they should therefore not be compared numerically.

4.4 Chloride diffusion behaviour

Fig. 7 shows the chloride concentration profiles with depth after 90 wetting–drying cycles (equivalent to 120 days) and the apparent chloride diffusion coefficients obtained by inverse Fick's law analysis. The prefabricated specimens exhibit a diffusion coefficient of $2.3 \times 10^{-12} \text{ m}^2/\text{s}$, which is

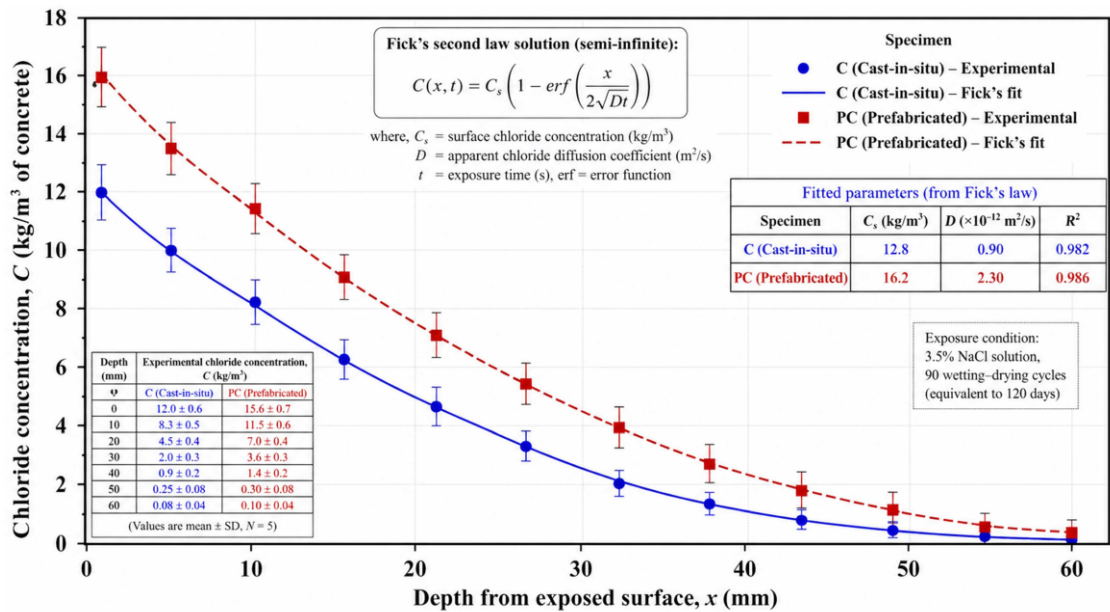


Figure 7. Chloride concentration profiles with depth after 90 wetting–drying cycles (120 days). Symbols: experimental results (mean $\pm$ SD, $N = 5$); lines: fitted curves using Fick's second law solution. The prefabricated specimens exhibit a higher diffusion coefficient ($2.30 \times 10^{-12} \text{ m}^2/\text{s}$) than cast-in-situ specimens ($0.90 \times 10^{-12} \text{ m}^2/\text{s}$), indicating accelerated chloride ingress along interfacial regions

about 2.5 times the value for cast-in-situ specimens ($0.9 \times 10^{-12} \text{ m}^2/\text{s}$). This confirms the presence of preferential chloride pathways in prefabricated joints, especially at the interfaces.

An increase in diffusion coefficient leads to rapid chloride accumulation at the level of the reinforcement, which results in earlier initiation of corrosion. This suggests that the diffusion process obeys Fick's law, and that permeability plays a crucial role in the rate of chloride ingress. In PC-DSS specimens, despite the chloride ingress being similar, the time to corrosion initiation was delayed because of the enhanced durability of DSS.

4.5 Time to corrosion initiation

The time to corrosion initiation differs significantly between specimen categories. Corrosion initiation in prefabricated specimens occurred at about 1.8 years (accelerated equivalent), while it occurred at 4.6 years for cast-in-situ specimens. This corresponds to almost a 60% reduction in time to initiation for prefabricated joints. The observed delay in PC-DSS specimens demonstrates the effectiveness of corrosion-resistant reinforcement in delaying initiation, even under high levels of chloride exposure.

4.6 Crack width development

The cracking behaviour is shown in Fig. 8. The crack widths of prefabricated joints were more than double those found in cast-in-situ joints.

An interesting feature is that cracking in prefabricated joints seemed to be delayed initially, but once initiated, the deterioration accelerated. This is indicative of an internal corrosion-product build-up before surface damage becomes visible. When the internal pressure reached the tensile strength of concrete, rapid cracking took place, resulting in rapid decay. This suggests a delayed onset of surface damage caused by internal corrosion products, typical of localised corrosion at interfaces. PC-DSS specimens exhibited fewer cracks, suggesting higher resistance to expansion due to corrosion.

The maximum crack width and Damage Localisation Index (DLI) for each specimen category are summarised in **Table 5**. Specimens with prefabricated joints show a much larger crack width and DLI than cast-in-situ specimens, suggesting a very concentrated damage pattern in the vicinity of the joints. The increased standard deviation in prefabricated specimens reflects the variability of interfacial defects and transport paths in the specimens, reinforcing the interface-controlled corrosion mechanism.

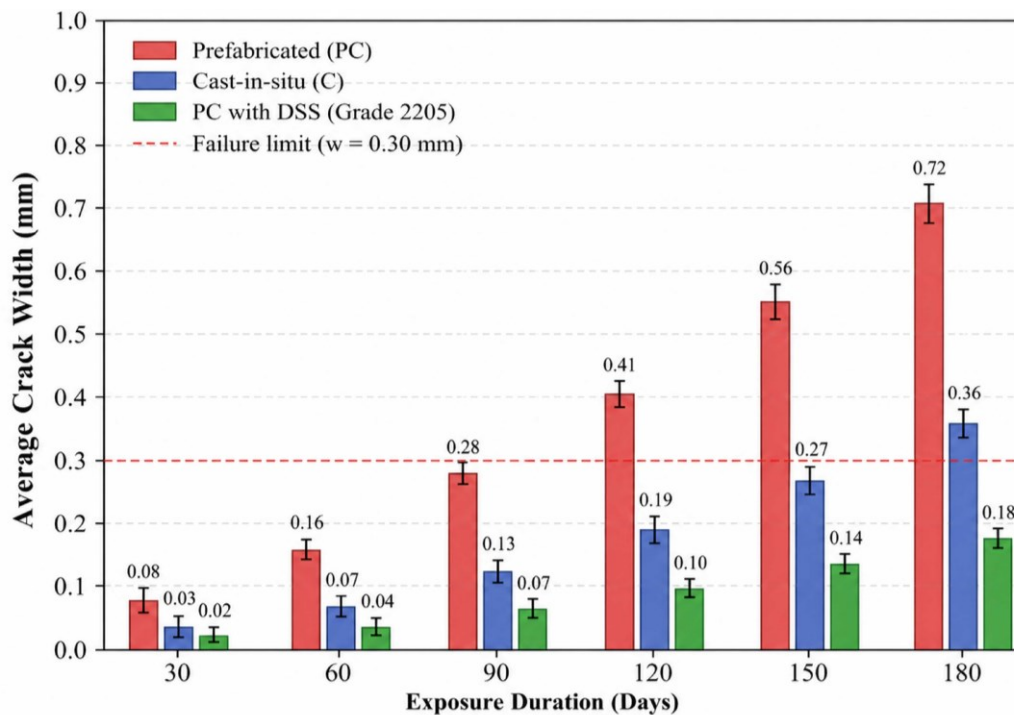


Figure 8. Average crack width versus exposure duration under cyclic wetting–drying in 3.5% NaCl solution for the three specimen categories. The red dashed line indicates the typical serviceability failure limit ($w = 0.30 \text{ mm}$). The PC specimens cross the failure limit between 60 and 90 days, while the C specimens cross it only after about 150 days. The PC-DSS specimens remain well below the limit throughout. Error bars: $\pm \text{SD}$, $N = 5$

Table 5. Crack width and damage localisation.

Specimen Type	Max Crack Width (mm)	SD (mm)	DLI = $M_{\text{interface}} / M_{\text{total}}$	Observation
C	0.18	0.003	0.28	Distributed damage
PC	0.42	0.006	0.62	Highly localised damage
PC-DSS	0.09	0.002	0.13	Minimal damage

4.7 Microstructural quantification

Microstructural analysis by CT scanning (Fig. 9) confirms that prefabricated joints have about 2–3 times more porosity than cast-in-situ joints. These discontinuities are predominantly located at the interface and confirm these defects as the pathway for chloride penetration. Such microstructural imperfections are responsible for the increased corrosion of prefabricated joints. The PC-DSS specimens had a comparable pore microstructure; however, the level of corrosion was minimal, suggesting that the choice of reinforcement plays a key role in the resulting deterioration.

4.8 Damage localisation index

To assess the corrosion-damage distribution pattern, the Damage Localisation Index (DLI) was calculated as the ratio of the steel mass loss occurring in the interfacial zone to the total steel mass loss. The DLI values for all categories are

summarised in Table 5. The prefabricated specimens had a DLI of 0.62, meaning that damage was very localised and concentrated around critical joint areas. The cast-in-situ specimens had a lower DLI of 0.28, indicating more even corrosion distribution. The use of corrosion-resistant reinforcement was seen to be beneficial in reducing localised deterioration, as the PC-DSS specimens had the lowest DLI (0.13).

4.9 Statistical validation

The experimental results were statistically validated. The mean steel mass loss for prefabricated specimens is 7.6%, with a standard deviation of 0.8% and a coefficient of variation of about 10.5%. The t-test between specimen types yielded a p-value of less than 0.01, which indicates that the observed differences between the specimens are statistically significant and not due to random scatter. The full statistical summary is reported in Table 6.

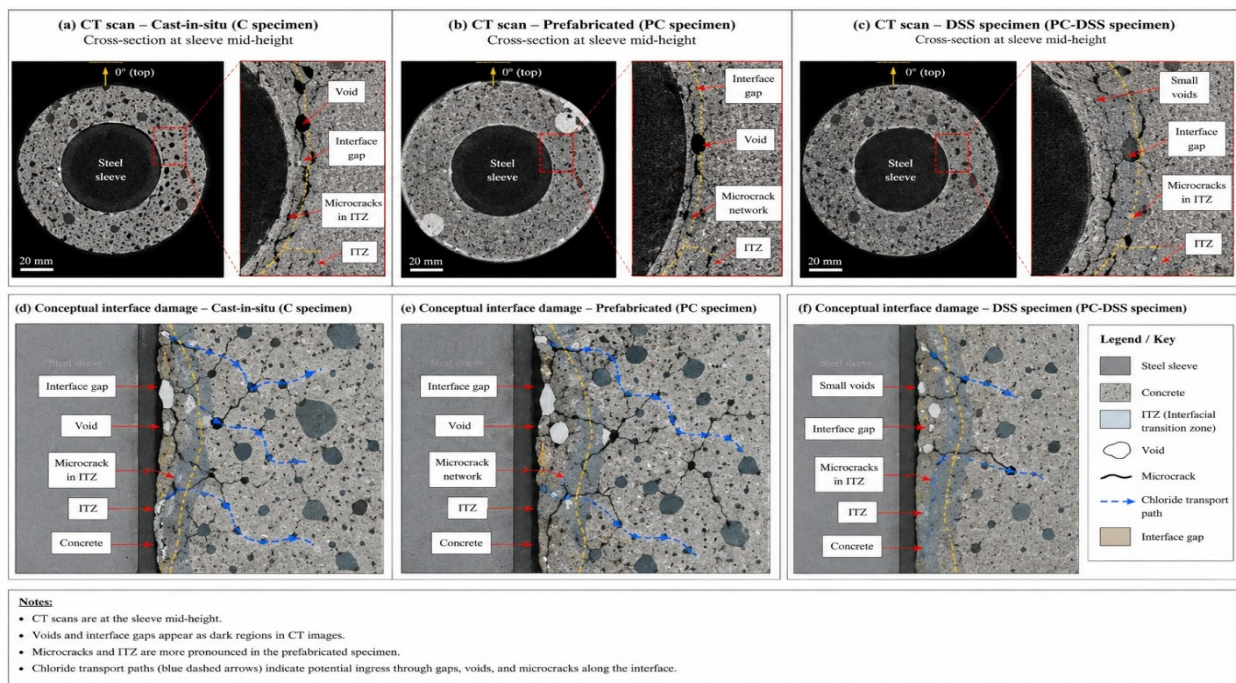


Figure 9. CT scan and conceptual interface damage diagrams of the three specimen categories. (a)–(c): CT scans of cast-in-situ (C), prefabricated (PC) and PC-DSS specimens at the sleeve mid-height cross-section. (d)–(f): conceptual interface damage diagrams highlighting voids, interface gaps, microcracks in the interfacial transition zone (ITZ), and chloride transport paths (blue dashed arrows). The PC specimen shows the densest microcrack network and the widest interface gaps, while the C specimen exhibits localised but discrete defects

Table 6. Statistical summary of corrosion results (PC specimens).

Parameter	Value
Mean Mass Loss (%)	7.6
Standard Deviation	0.8
Coefficient of Variation (%)	10.5
p-value (t-test)	< 0.01
Interpretation	Statistically significant difference

4.10 Interface-controlled corrosion mechanism

The findings suggest that the corrosion process in prefabricated joints cannot be explained by homogeneous diffusion models. Rather, interfaces serve as fast diffusion pathways, resulting in chloride build-up and heterogeneities that lead to pit formation at the interfaces. The elevated DLI values and their variability also support the view that corrosion is driven by the interfacial regions. The multiple interfaces create distinct transport routes that affect chloride penetration and subsequent corrosion [2], [6]. The increased diffusion coefficients in prefabricated specimens verify that interfaces provide fast pathways for chloride ingress due to their lower resistance. This validates the notion of interface-controlled corrosion, where the interfacial discontinuities control the transport and electrochemical reaction [4]. Consequently, corrosion starts earlier and develops more rapidly in prefabricated concrete than in cast-in-situ concrete.

4.11 Localisation of corrosion and damage evolution

The most noteworthy finding of this study is the highly localised corrosion in prefabricated joints. The higher DLI (≈ 0.62) suggests that damage is preferential around critical locations including sleeves and construction imperfections. This behaviour is explained by the spatial distribution of chlorides, moisture and oxygen, which results in an electrochemical gradient and pitting. This is in contrast to the relatively uniform corrosion front observed in cast-in-situ specimens, where loss of reinforcement cross-section is more uniform. This is also reflected in the observed cracking behaviour, where damage is internally confined initially and then suddenly appears as surface cracking when the internal stresses exceed the tensile strength of concrete [12].

4.12 Influence of interfacial characteristics on transport behaviour

Interfacial characteristics play a key role in controlling chloride transport and corrosion. Microstructural studies showed greater porosity, microvoids and poor bonding at the concrete–grout and grout–steel interfaces [13]. Such characteristics result in increased permeability and promote uneven chloride transport. The findings suggest that minor defects in interfacial bonding can lead to high transport rates and increased corrosion rates. This underlines the importance of considering non-uniform material properties when modelling durability and of incorporating interfacial effects when predicting the performance of prefabricated structures [6].

4.13 Effectiveness of duplex stainless steel as a targeted mitigation strategy

The use of duplex stainless steel (Grade 2205) in key areas was very effective in reducing corrosion [8]. Although the PC-DSS specimens were exposed to the same environment and experienced a similar degree of chloride penetration, they showed delayed corrosion onset and much less damage. This can be explained by the protective passive film and excellent pitting resistance of duplex stainless steel in chloride-rich environments [11]. Crucially, the results show that strategic incorporation of corrosion-resistant materials in critical interfacial areas is more effective than complete replacement, providing a cost-effective approach to improve durability.

4.14 Implications for design and durability of prefabricated systems

Our study has significant implications for the design and durability of prefabricated concrete systems. Current design considerations based on bulk properties could overlook durability risks because they fail to consider the impact of interfaces. The accelerated corrosion initiation and higher corrosion rates shown here indicate that prefabricated systems may have a reduced service life if interfaces are not properly managed [1]. Therefore, durability design should focus on interfaces, using better material compatibility, improved construction processes and targeted reinforcement strategies. Furthermore, performance-based durability models incorporating interfacial transport processes are required to improve service-life predictions [14].

4.15 Mitigation Strategies

The high DLI values (≈ 0.62) and localised corrosion near the sleeve interfaces suggest that mitigation strategies should be focused on the interface. The experimental results indicate that the corrosion of prefabricated joints is controlled by interface defects, pathways for the ingress of chlorides, and local exposure of the reinforcement, especially in areas such as the sleeve interface and pump/vent holes [4], [10]. A multi-level and integrated approach should therefore be adopted, including material properties, interface engineering, design considerations, quality control, and long-term monitoring.

One of the core approaches is to employ dense and high-performance materials in the joint region. The use of high-performance concrete (HPC) and dense, non-shrinkable grouting materials helps to minimise chloride diffusion through pore refinement [1]. The incorporation of supplementary cementitious materials such as fly ash, silica fume and ground granulated blast furnace slag is effective in improving durability by reducing permeability and the time to reach a critical chloride concentration at the reinforcement. Another critical factor is interface engineering, since corrosion in prefabricated structures is interface-dominated [4]. Improving the interface between concrete, grout and reinforcement is crucial to prevent the formation of micro-gaps and weak interfaces. This involves roughening the surface of the precast components, cleaning before grouting, and using appropriate bonding agents. Material compatibility also minimises shrinkage and thermal incompatibilities, which are two major causes of joint cracking.

Joint design also helps reduce corrosion. Greater cover thickness in key areas, such as around transverse bars, helps protect against aggressive exposure. Preventing contact of the reinforcement with weak zones, like sleeve ends, can help avoid localised corrosion. Finally, redesigning or sealing pump and vent holes after grouting can restrict direct ingress of chlorides.

Proper construction of prefabricated joints is also critical. Poor grouting, residual voids, and air pockets are typical causes of premature failure. Quality control must therefore be maintained during construction through correct grout injection and compaction, and by checking for full filling of sleeves. Controlling the grouting process and monitoring joint installation also help to ensure a better joint [5].

Corrosion-resistant reinforcement materials offer an effective and cost-conscious solution. Specifically, duplex stainless steel (Grade 2205) has excellent resistance to chloride attack owing to its stable passive film and pitting

resistance [8], [11]. Its localised application in areas prone to damage, such as stirrups at the sleeve junction, can delay the onset of corrosion and minimise localised damage, thus extending the lifetime of prefabricated structures.

Long-term monitoring and maintenance are key to maintaining structural integrity. The use of embedded sensors to monitor moisture, chloride and corrosion potential allows real-time assessment of structural integrity. Together with predictive algorithms, these monitoring systems can enable early detection and mitigation of corrosion, leading to longer service life. Overall, control of corrosion in prefabricated joints requires an integrated strategy that includes material development, better interface conditions, proper design, sound construction practice and new monitoring methods [8].

4.16 Limitations and future research

Although the research offers insights into the corrosion of prefabricated joints, some limitations must be considered. The laboratory-based experimental programme involved an accelerated exposure, which may not fully replicate long-term field conditions. Moreover, the model adopted to describe chloride transport is based on simplifications and does not explicitly quantify interfacial parameters. The next steps for research include the creation of refined multi-scale models considering interfacial features, verification of the results under field conditions, and the development of other mitigation measures, such as coatings and smart sensor technologies [12].

5 Conclusions

This research confirms that the corrosion performance of prefabricated RC beam–column joints with grouted sleeve connections is primarily influenced by interfacial properties. This conclusion was confirmed through half-cell potential, visual damage, loss of mass in the steel and observations of interface deterioration after cyclic chloride exposure. The experimental findings demonstrate that prefabricated specimens have a much earlier onset of corrosion (reduced by 40–60% relative to cast-in-situ joints) because of rapid chloride penetration through the interfaces. Prefabricated joints exhibited a corrosion rate almost 2.8 times higher, with a chloride diffusion coefficient 2.5 times larger, confirming preferential pathways for chloride ingress. Additionally, corrosion damage was highly concentrated, as shown by a Damage Localisation Index (DLI) of approximately 0.62, with damage concentrated around sleeve connections and construction-related discontinuities. This led to more severe cracking, with crack widths over twice those in conventional specimens. By contrast, the use of duplex stainless steel (Grade 2205) in key areas led to a substantial delay in corrosion initiation and less severe damage, confirming the potential of selective reinforcement to mitigate corrosion. In summary, the results confirm the interface-controlled nature of corrosion and highlight the need for interface-oriented design, enhanced construction practices, and localised application of corrosion-resistant materials to improve the sustainability and lifespan of prefabricated concrete structures.

Acknowledgment

The authors express their sincere gratitude to the Department of Civil Engineering, Noida International University, for providing facilities and technical support throughout the research.

Funding

This research received no specific grant from any funding agency in the public, commercial, or not-for-profit sectors.

Conflict of Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data Availability Statement

The datasets generated and/or analysed during the current study are available from the corresponding author on reasonable request.

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Prediction of slump value of concrete containing coconut shell aggregate using artificial neural networking

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Article history

Received: 04 May 2026

Received in revised form:

22 June 2026

Accepted: 23 June 2026

Available online: 13 July 2026

Keywords

Coconut Shell Aggregate (CSA) concrete,
slump prediction,
artificial neural network,
machine learning,
sustainable concrete

ABSTRACT

The use of Coconut Shell (CS) as a sustainable aggregate in concrete provides an effective alternative to conventional materials; however, it significantly influences workability characteristics. This study presents the development of an Artificial Neural Network (ANN) model for predicting the slump of Coconut Shell Aggregate (CSA) concrete. A dataset comprising 65 samples was compiled from published literature, considering cement, fine aggregate, coarse aggregate, CSA, water–cement ratio, and percentage replacement of CSA as input variables, with slump as the output parameter. The dataset was standardized and divided into training (80%) and testing (20%) subsets. A feedforward ANN model with a single hidden neuron and hyperbolic tangent activation function was implemented in Python using TensorFlow. The model was trained using the Adam optimizer with Mean Squared Error loss, and Early Stopping was employed to improve generalization and prevent overfitting. The model achieved a coefficient of determination of $R^2 = 0.7150$ for training and $R^2 = 0.6046$ for testing. The corresponding error values were Root Mean Square Error (RMSE) of 21.920 and 27.367, and Mean Squared Error (MAE) of 15.551 and 18.298 for training and testing, respectively. Validation using an independent mix resulted in a prediction error of 3.97%, indicating good predictive capability. Model interpretability was assessed using the Garson Algorithm, which identified fine aggregate (0.5879) as the most influential parameter, followed by cement (0.1645) and water-related parameter (0.1558), while other variables showed comparatively lower influence. Connection Weight analysis further confirmed these trends. The results demonstrate that the ANN model provides a reliable and interpretable tool for predicting workability in sustainable concrete systems.

1 Introduction

The slump test is one of the most common tests used to assess the workability of fresh concrete mixtures. It is conducted in both laboratories and at construction sites where concrete is placed. The test provides an indication of the consistency and fluidity of the concrete mix. In this test, fresh concrete is placed in a standard slump cone in three layers, and each layer is compacted with a tamping rod. After filling the cone, it is lifted vertically, and the concrete subsides or “slumps.” The difference between the original height of the cone and the height of the slumped concrete is measured in millimeters and is called the slump value of the concrete mix [1]. The slump value indicates the workability of the concrete mix. A higher slump value shows greater workability and ease of placement with less compaction effort. Therefore, a minimum slump value is recommended for placing concrete in structural elements such as beams, slabs, columns, and footings as per IS 456:2000 [2]. The

Coconut Shell Aggregates (CSA) exhibit a highly porous structure and significantly higher water absorption capacity than conventional crushed stone aggregates. Due to this characteristic, a considerable amount of mixing water is absorbed by the Coconut Shell (CS) particles, reducing the free water available for cement hydration and lubrication within the concrete mix. Consequently, the workability of fresh concrete may decrease, leading to difficulties in mixing, transportation, placement, and compaction. Inadequate workability can also result in poor consolidation and the formation of voids within the hardened concrete. Therefore, workability evaluation and appropriate mix design adjustments, such as controlling the water-to-cement ratio or pre-soaking the CSA, are essential to ensure satisfactory fresh and hardened concrete performance when CSA are used as partial or full replacements for conventional coarse aggregates [3&4].

The construction industry is facing an increasing demand for natural aggregates, which occupy 70–80% of the total

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volume of a concrete mixture. Continuous extraction of natural aggregates from river deposits and rock quarries leads to resource depletion. In recent years, the use of alternative materials in concrete production has become a sustainable approach to concrete manufacturing [5&6]. In most reported studies, CS has been utilized as a replacement for conventional coarse aggregate, while natural fine aggregate is retained in the concrete mixture.

The CSA in concrete has been reported to achieve compressive strengths in the range of 19–20 MPa after 28 days of curing, making it suitable for several structural and non-structural applications. However, the mechanical performance of such concrete is strongly influenced by the replacement ratio and mix design parameters. The CS are primarily used as a partial replacement for coarse aggregates due to their lower density, higher porosity, and greater water absorption characteristics compared with natural aggregates. At moderate replacement levels, acceptable strength can be achieved without significant increases in cement content, whereas excessive replacement may result in strength reduction because of weaker aggregate–paste interaction and increased void content. Therefore, the selection of an appropriate replacement ratio is essential to balance strength, workability, and sustainability requirements. Furthermore, workability and compressive strength are closely related, as inadequate workability may lead to poor compaction and increased porosity, ultimately affecting the strength and durability of concrete. Consequently, accurate prediction of slump is important for optimizing fresh concrete performance and ensuring the desired mechanical properties of CSA concrete [5]. The CSA in concrete creates a better bonding with cement paste due to its rough surfaces [7].

The use of CS in concrete improves thermal insulation and reduces the density of concrete, leading to more sustainable and energy-efficient construction. Therefore, the utilization of CSA in concrete represents a sustainable approach by converting agricultural waste into a useful construction material while also conserving natural resources [8&9].

In recent years, machine learning techniques have gained significant attention in civil engineering for predicting the properties of concrete, including compressive strength, slump, and durability. Traditional experimental methods used to determine these properties are often time-consuming, labor-intensive, and costly. Machine learning models provide an efficient alternative by establishing relationships between input parameters and concrete properties based on available datasets. These models can analyze complex nonlinear interactions among materials in concrete mixtures and generating accurate predictions with reduced experimental effort, thereby improving the efficiency of concrete mix design and material optimization [10 &11]. Among various machine learning techniques, Artificial Neural Networks (ANN) are one of the most widely used methods. They are structured like the human brain and consist of interconnected neurons organized into input, hidden, and output layers. The network learns patterns within the given dataset through a training process and can identify the relationship between the input and output variables. [12].

The ANN models have been widely applied in concrete technology due to their ability to capture complex nonlinear relationships among mixture constituents and fresh concrete properties. Several studies have demonstrated that ANN-based models can accurately predict workability characteristics, particularly the slump value of concrete mixtures, using input parameters such as cement content,

water-to-cement ratio, aggregate proportions, admixture dosage, and supplementary cementitious materials [13–16]. The capability of ANN models to learn from experimental datasets makes them a reliable tool for predicting concrete workability and reducing the need for extensive laboratory testing.

Therefore, the ANN models can effectively predict workability characteristics, particularly the slump value of concrete mixtures. Therefore, the application of machine learning techniques offers an efficient and reliable approach for enhancing the optimization and sustainability of concrete mix design having the CSA.

The replacement of conventional coarse aggregate with CSA was considered, while the natural fine aggregate remained unchanged. The primary objective of incorporating CSA is to reduce the consumption of natural coarse aggregate and promote the utilization of agricultural waste materials, thereby enhancing the sustainability of concrete production. Furthermore, the selected replacement levels are intended to ensure that the concrete possesses adequate workability for practical applications in structural members such as beams, slabs, columns, and foundations [17–26].

The present study is to develop and validate an artificial neural network model for predicting the slump of CSA concrete using datasets compiled from published experimental investigations (Literature Review) and validated through independent experimental testing.

2 Materials and data compilation

2.1 Data sources

The experimental data were collected from previously published studies, and these data are summarized and presented in Table 1. The dataset variables cement, fine aggregate, coarse aggregate, coconut shell aggregate (CSA), water–cement (w/c) ratio, slump, and percentage of CSA replacement were used.

Datasets 1–20 were obtained from Reference 17; datasets 21–23 were sourced from Reference 18; datasets 24–29 were taken from Reference 19; and dataset 30 was collected from Reference 20. Furthermore, datasets 31–37 were obtained from Reference 21; datasets 38–41 were sourced from Reference 22; datasets 42–46 were taken from Reference 23; datasets 47–51 were obtained from Reference 24, data sets 52–57 from Reference 25, datasets 58–61 were obtained from Reference 26 and datasets 62–65 were obtained from Reference 27.

The dataset was divided into two phases: training (80% of the total dataset) and testing (20% of the total dataset).

The statistical summary of the dataset indicates a wide variation in mix proportions and workability characteristics. Cement content ranges from 300 to 510 kg/m³, while fine aggregate varies between 531 and 981 kg/m³. The coarse aggregate and CSA contents exhibit significant variability due to different replacement levels, ranging from 0 to 1263 kg/m³ and 0 to 1176 kg/m³, respectively. The water–cement ratio ranges from 0.38 to 0.72, reflecting variations in mix consistency.

The slump values vary from 0 to 150 mm, indicating a broad spectrum of workability conditions. The percentage replacement of CSA ranges from 0% to 100%, ensuring that the dataset captures both conventional and fully replaced mixes. This variability enhances the robustness of the ANN model by enabling it to learn diverse material behavior patterns.

Table1. Experimental data collected from the literatures

Sample No.	Cement (kg/m ³)	Fine aggregate (kg/m ³)	Coarse aggregate (kg/m ³)	Coconut Shell Aggregate (CSA) (kg/m ³)	water/ cement ratio	Percentage (%) of Replacement of CSA	Slump (mm)	28 th Day Cubical Compressive Strength (MPa)
1.	300	981	0	402	0.72	100	10	4.95
2.	400	820	0	336	0.55	100	25	9.81
3.	425	820	0	336	0.50	100	15	13.24
4.	450	824	0	338	0.45	100	25	13.49
5.	480	658	0	360	0.51	100	110	10.30
6.	480	802	0	331	0.42	100	65	15.20
7.	480	730	0	360	0.42	100	50	16.19
8.	480	768	0	384	0.44	100	50	16.68
9.	480	768	0	384	0.42	100	5	17.66
10.	480	768	0	336	0.42	100	30	18.15
11.	510	750	0	332	0.42	100	5	26.70
12.	510	928	0	281	0.38	100	0	23.40
13.	510	887	0	281	0.42	100	5	16.72
14.	510	801	0	281	0.48	100	140	13.38
15.	510	867	0	306	0.38	100	0	19.50
16.	510	816	0	306	0.42	100	0	16.16
17.	510	734	0	306	0.48	100	40	13.38
18.	510	806	0	332	0.38	100	0	27.20
19.	510	750	0	332	0.42	100	5	26.70
20.	510	673	0	332	0.48	100	150	14.50
21.	450	531	1238	0	0.40	0	60.5	46.7
22.	450	531	1176	62	0.40	5	50	29.6
23.	450	531	1114	124	0.40	10	25	7.4
24.	403	672	1260	0	0.48	0	72	27.58
25.	403	672	1134	126	0.48	10	66	23.46
26.	403	672	1008	252	0.48	20	60	20.95
27.	403	672	882	378	0.48	30	52	18.94
28.	403	672	756	504	0.48	40	46	17.06
29.	403	672	630	630	0.48	50	37	15.48
30.	340	690	952	21	0.57	5	83	38
31.	395	607.43	1213.82	0	0.50	0	85	25.65
32.	395	607.43	1153.13	60.69	0.50	5	79	25.20
33.	395	607.43	1092.43	121.382	0.50	10	73	24.25
34.	395	607.43	1031.74	182.07	0.50	15	67	22.92
35.	395	607.43	971.05	242.76	0.50	20	61	20.19
36.	395	607.43	910.36	303.45	0.50	25	57	19.52
37.	395	607.43	849.68	364.15	0.50	30	55	17.28
38.	300	750	1170	0	0.60	0	25	22.33
39.	300	750	1053	117	0.60	10	23	13.62
40.	300	750	994.5	175.5	0.60	15	22	9.38
41.	300	750	936	234	0.60	20	20	7.15
42.	403	672	1260	0	0.50	0	85	20.27
43.	403	672	1228.5	31.5	0.50	2.5	61	19.71
44.	403	672	1197	63	0.50	5.0	67	19.53
45.	403	672	1165.5	94.5	0.50	7.5	73	19.08
46.	403	672	1134	126	0.50	10	79	18.91
47.	383	536.2	957.50	0	0.5	0	75	34.66
48.	383	536.2	838.77	118.73	0.5	10	61	25.61
49.	383	536.2	612.80	256.61	0.5	21	55	19.74
50.	383	536.2	287.25	356.19	0.5	29	52	17.78
51.	383	536.2	229.80	383	0.5	31	50	16.52
52.	384.35	580.4	0	1176.1	0.5	0	26	30.64
53.	384.35	580.4	940.88	235.22	0.5	20	26	22.33
54.	384.35	580.4	705.66	470.44	0.5	40	27	18
55.	384.35	580.4	407.44	705.66	0.5	60	28	13.56
56.	384.35	580.4	235.22	940.88	0.5	80	29	11.42
57.	384.35	580.4	0	1176.1	0.5	100	31	8.33
58.	405	667	1263	0	0.44	0	40	34.10

59.	405	667	1136.7	126.3	0.44	10	30	28.53
60.	405	667	1010.4	252.6	0.44	20	35	22.32
61.	405	667	884.1	378.9	0.44	30	42	20.12
62.	419	976	958	0	0.55	0	115	25.8
63.	419	976	919.7	38.3	0.55	4	116	20.91
64.	419	976	881.4	76.6	0.55	8	113	14.65
65.	419	976	843.0	114.9	0.55	12	110	9.40
Min	300	531	0	0	0.38	0	0	4.95
Max	510	981	1263	1176	0.72	100	150	46.70

2.2 Input variables

Six input parameters were considered for the development of the predictive model, selected based on their known influence on concrete workability:

Cement content (kg/m^3): Represents the binding material governing paste volume and consistency.

Fine aggregate (kg/m^3): Influences packing density and water demand, significantly affecting slump.

Coarse aggregate (kg/m^3): Contributes to the overall skeleton of the mix and affects flow characteristics.

Coconut shell aggregate (CSA) (kg/m^3): Acts as a lightweight alternative aggregate, altering absorption and surface texture properties.

Water–cement ratio: A critical parameter controlling the fluidity and consistency of concrete.

Percentage replacement of CSA (%): Indicates the proportion of coconut shell used in place of conventional coarse aggregate, directly impacting workability behavior.

These variables were selected to capture the combined effects of material composition and replacement strategy on the slump of concrete.

2.3 Output parameter

The output parameter considered in this study is the slump value (mm), which serves as a standard measure of concrete workability. Slump values were obtained from the compiled literature dataset, ensuring consistency in testing procedures. The slump parameter was chosen due to its practical significance in assessing the ease of mixing, placing, and compaction of concrete in engineering applications.

3 Artificial neural network model development

3.1 ANN architecture

A feedforward Artificial Neural Network (ANN) was developed to model the relationship between concrete mix constituents and slump. The network was implemented using the TensorFlow–Keras framework [28] in a sequential configuration. The input layer consists of six neurons corresponding to the selected variables: cement, fine aggregate, coarse aggregate, coconut shell aggregate (CSA), water–cement ratio, and percentage replacement of CSA.

The hidden layer comprises a single neuron with a hyperbolic tangent ($\tanh$) activation function. The $\tanh$ function introduces nonlinearity and enables the network to capture complex interactions among the input variables. Although the architecture is relatively simple, it is adequate for capturing the underlying trends in the available dataset while minimizing the risk of overfitting.

The output layer contains a single neuron with a linear activation function, which produces the predicted slump value as a continuous output. The overall architecture can therefore be described as a 6–1–1 network structure. The ANN for CSA concrete slump prediction as shown in Figure 1.

3.2 Training strategy

Prior to training, the input and output data were standardized using the Standard Scaler method [29] to ensure zero mean and unit variance, thereby improving numerical stability and convergence of the model.

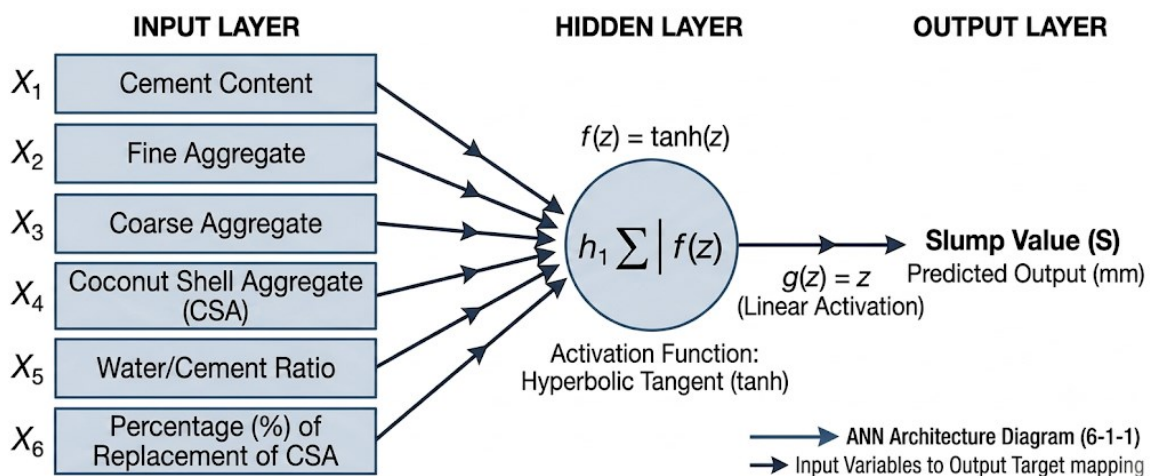


Figure 1. Schematic diagram of the 6–1–1 ANN for slump value forecasting

The ANN was trained using the Adam optimization algorithm [30] with a learning rate of 0.001. Adam combines the advantages of adaptive learning rates and momentum, making it suitable for nonlinear regression problems involving small to moderate datasets. The loss function employed was Mean Squared Error (MSE), which minimizes the squared differences between predicted and actual slump values.

To enhance model generalization and prevent overfitting, Early Stopping was implemented by monitoring the validation loss during training. The training process was terminated when no improvement in validation loss was observed for a predefined number of epochs, and the best-performing model weights were restored. This approach ensures that the model retains optimal predictive performance on unseen data.

4 Model performance evaluation

4.1 Training and testing results

The predictive performance of the developed ANN model was evaluated using standard statistical indicators, namely the coefficient of determination (R^2), Root Mean Square Error (RMSE), and Mean Absolute Error (MAE). These metrics provide a comprehensive assessment of model accuracy by quantifying both the strength of correlation and the magnitude of prediction errors.

4.2 Validation with experimental data

To further assess the practical applicability of the developed model, validation was performed using an independent concrete mix not included in the training or testing datasets. The predicted slump value obtained from the ANN model was compared with the experimentally measured slump. The experimental mix proportions and the measured slump value is shown in the Table 2.

The percentage error was calculated using the following expression:

$$\% \text{ Error} = \left(\frac{|S_{\text{actual}} - S_{\text{predicted}}|}{S_{\text{actual}}} \right) \times 100 \quad (1)$$

where S_{actual} and $S_{\text{predicted}}$ represent the measured and predicted slump values, respectively. The validation results indicated a prediction error of 3.97%, demonstrating good agreement between the experimental and predicted values. This low error confirms the reliability of the ANN model in estimating slump for new mix proportions.

5 Sensitivity and interpretability analysis

5.1 Garson algorithm

To address the interpretability of the developed ANN model, the relative importance of input variables was evaluated using the Garson Algorithm. This method

quantifies the contribution of each input parameter by partitioning the connection weights between the input, hidden, and output layers [31]. The algorithm computes the relative importance based on the absolute values of the product of input–hidden and hidden–output weights, followed by normalization.

The relative importance I_i of the i^{th} input variable is expressed as:

$$I_i = \frac{\sum_{j=1}^{n_h} |w_{ij} * w_j|}{\sum_{i=1}^n \sum_{j=1}^{n_h} |w_{ij} * w_j|} \quad (2)$$

where w_{ij} represents the weight between the i^{th} input neuron and the j^{th} hidden neuron, w_j is the weight between the j^{th} hidden neuron and the output neuron, n_h is the number of input variables, and n_h is the number of hidden neurons.

5.2 Connection weight analysis

To further examine the influence of input variables, Connection Weight Analysis was performed. This method evaluates both the magnitude and direction (positive or negative) of the relationship between inputs and the output by computing the combined effect of weights across the network [31].

The connection weight for each input variable is determined using:

$$CWi = \sum_{j=1}^{n_h} (w_{ij} * w_j) \quad (3)$$

The resulting values were normalized to facilitate comparison across inputs. Positive values indicate a direct relationship with slump, whereas negative values represent an inverse relationship.

6 Results

6.1 Model performance metrics

The predictive performance of the developed model was assessed using the Coefficient of Determination (R^2), Root Mean Square Error (RMSE), and Mean Absolute Error (MAE) for both the training and testing datasets. For the training dataset, which comprised 80% of the total data, the model achieved an R^2 value of 0.7150, indicating that approximately 71.50% of the variation in the target variable was explained by the model. The corresponding RMSE and MAE values were 21.920 mm and 15.551 mm, respectively. For the testing dataset, representing the remaining 20% of the data, the model obtained an R^2 value of 0.6046, demonstrating a satisfactory predictive capability on unseen data. The RMSE and MAE values for the testing dataset were 27.367 mm and 18.298 mm, respectively. Although the testing performance showed a slight reduction compared to the training results, the model maintained reasonable prediction accuracy and generalization ability, indicating its suitability for estimating the response variable.

Table 2. Experimental mix proportion for concrete with CSA and its slump Value

Cement (kg/m ³)	Fine aggregate (kg/m ³)	Coarse aggregate (kg/m ³)	Coconut Shell aggregate (kg/m ³)	water/cement ratio	Slump (mm)
335	528	491	242	0.5	110

6.2 Experimental validation result

To evaluate the practical applicability and predictive capability of the developed model on independent data a validation mix was tested and its experimentally (Shown in Table 2) measured slump value was compared with the model prediction. As presented in the Table 3 the validation mix exhibited an actual slump (S_{actual}) of 110 mm, while the model predicted a slump ($S_{\text{predicted}}$) of 114.36 mm. The resulting absolute error between the predicted and actual values was only 4.36 mm, corresponding to a percentage error of 3.97%. This relatively low prediction error demonstrates the robustness and reliability of the developed model in estimating slump values for new concrete mixtures, confirming its suitability for practical applications and validating its generalization performance beyond the training and testing datasets.

6.3 Sensitivity analysis

6.3.1 Garson algorithm

The relative importance of the input variables influencing the slump value was evaluated using the Garson weight partitioning algorithm. The analysis revealed that fine aggregate was the most influential parameter, with a relative importance value of 0.5879, contributing 58.79% to the model output. Cement was identified as the second most significant factor, with a contribution of 16.45% (relative importance value of 0.1645), followed closely by the water–cement ratio, which contributed 15.58% (relative importance value of 0.1558). Coarse aggregate exhibited a comparatively lower influence, accounting for 5.99% of the total contribution with a relative importance value of 0.0599. In contrast, coconut shell aggregate (CSA) and the percentage replacement of CSA showed minimal effects on slump prediction, contributing only 1.65% and 1.54%, respectively, with relative importance values of 0.0165 and 0.0154. These results indicate that fine aggregate, cement, and the water–cement ratio are the dominant factors governing the slump behavior of coconut shell aggregate concrete, while CSA-related parameters have a relatively minor influence within the range of data considered in this study.

6.3.2 Connection weight

The Connection Weight Analysis was conducted to determine the directional influence of the input variables on the slump of CSA concrete. The results indicated that fine aggregate exhibited a strong positive influence on slump, suggesting that an increase in fine aggregate content leads to higher slump values and improved workability. Similarly, the water–cement ratio showed a positive relationship with

slump, indicating that higher water–cement ratios enhance the fluidity of the concrete mixture and consequently increase slump. In contrast, cement content demonstrated a negative influence on slump, implying that an increase in cement content tends to reduce workability and decrease slump under the conditions considered in this study. Furthermore, coconut shell aggregate (CSA) content and the percentage replacement of CSA were found to have relatively minor effects on slump compared with the other input variables, indicating a limited influence on workability within the range of mixtures investigated. These findings provide valuable insight into the role of individual mix constituents in governing the slump behavior of coconut shell aggregate concrete.

7 Discussion

The developed ANN model demonstrates a satisfactory ability to predict the slump of CSA concrete, as reflected by the obtained performance metrics. The analysis of input parameters indicates that fine aggregate exerts the most significant influence on slump, as confirmed by the Garson Algorithm (relative importance = 0.5879). This observation is consistent with the fundamental role of fine aggregate in controlling particle packing, surface area, and water demand, all of which directly affect workability. The water–cement ratio also shows a notable contribution, highlighting its importance in governing the fluidity and consistency of the concrete mix. Similarly, cement content influences the paste volume, thereby affecting lubrication between particles. In contrast, coarse aggregate, CSA, and their percentage replacement exhibit comparatively lower direct influence within the studied range. This suggests that while these parameters affect the overall mix behavior, their impact on slump is secondary compared to fine aggregate and water-related properties. The observed trends are generally consistent with findings reported in the literature, where workability is primarily governed by paste characteristics and fine aggregate content. The inclusion of CSA introduces variability due to its higher water absorption and irregular surface texture; however, its direct contribution to slump appears limited when compared to other parameters. Despite these promising results, the model has certain limitations. The ANN architecture employed is relatively simple, consisting of a single hidden neuron, which may restrict its ability to capture highly complex relationships. Additionally, the dataset is limited to 65 samples collected from different sources, which may introduce variability due to differences in experimental conditions. Therefore, the model performance may be improved with a larger and more consistent dataset, as well as a more optimized network structure.

Table 3. Comparison of experimental and predicted slump value

Mix ID	Actual Slump (mm)	Predicted Slump (mm)	Error (mm)	% Error
Validation Mix	110	114.36	4.36	3.97

8 Conclusions

This study presents the development of an Artificial Neural Network (ANN) model for predicting the slump of CSA concrete. Based on the results obtained, the following conclusions can be drawn:

- The ANN model achieved satisfactory predictive performance, with $R^2=0.7150$ for training and $R^2=0.6046$ for testing, indicating its capability to capture the relationship between mix constituents and slump.
- Error metrics (RMSE and MAE) demonstrate acceptable accuracy for practical engineering applications.
- Validation using an independent mix resulted in a low prediction error of 3.97%, confirming the reliability of the model.
- Sensitivity analysis revealed that fine aggregate is the most influential parameter, followed by cement and water-related properties, while CSA and its replacement percentage have relatively lower direct impact.
- The integration of Garson Algorithm and Connection Weight analysis improves model interpretability, addressing the "black box" limitation of ANN.

CRedit authorship contribution statement

B. Ranjith Babu: Conceptualization, Data curation, Investigation, Methodology, Software, Formal analysis, Validation, Writing – original draft, Writing – review & editing.

Declaration of competing interest

The author declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgement

This research did not receive any specific grant from funding agencies in the public, commercial, or not-for-profit sectors.

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Comparative performance evaluation of cement-stabilized Ota laterite blocks and conventional sandcrete for non-load-bearing applications in tropical climate

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Article history

Received: 23 June 2026

Received in revised form:

10 July 2026

Accepted: 11 July 2026

Available online: 24 July 2026

Keywords

cement-stabilized laterite,
sandcrete blocks,
compressive strength,
durability,
thermal conductivity,
predictive modeling,
tropical construction,
statistical validation,
sustainable materials,
low-cost housing

ABSTRACT

This study compares cement-stabilized laterite blocks (10% OPC) and conventional sandcrete blocks (1:6 ratio) for tropical construction. Seventy-two specimens were tested for compressive strength, density, water absorption, durability, and thermal conductivity. Sandcrete blocks achieved 2.68 ± 0.68 N/mm² at 28 days, satisfying the Nigerian Industrial Standard NIS 87:2000 load-bearing threshold (≥ 2.50 N/mm²), whereas laterite blocks reached 1.35 ± 0.09 N/mm². Laterite blocks exhibited lower production variability (coefficient of variation, CV = 6.5% vs. 25.3%), superior durability under wet-dry cycling (2.59% vs. 8.22% mass loss; significance $p < 0.001$), and lower thermal conductivity (0.343 vs. 1.222 W/m·K; $p < 0.001$). Both materials complied with water absorption limits. An exploratory power-law model was fitted to project strength at variable cement contents. The results support using laterite blocks for protected non-load-bearing applications where thermal performance and durability are prioritized, contributing to sustainable affordable housing in tropical climates.

1 Introduction

Construction emits approximately 39% of the world's CO₂, and Portland cement accounts for around 5-8% of the world's total anthropogenic carbon dioxide [1]. In Nigeria, the largest country in Africa by population, there is an over 28 million housing deficits with an annual shortfall of more than 700,000 [2]. The problem with affordability of conventional houses is further exacerbated by the fact that 60%-70% of total construction costs in developing nations go to materials [3]. Sand extraction as an aggregate material for construction has become an urgent environmental problem resulting in destroying riverbanks, habitat destruction and draining water tables in tropical areas worldwide [4].

For many decades, sandcrete blocks made up of Portland cement, sharp sand, and water in a typical volume ratio of 1:6 to 1:8 have been used in the construction of walls in Nigeria [5]. According to the Nigerian Industrial Standard NIS 87:2000, the minimum compressive strengths required are 2.50 N/mm² for load-bearing blocks (those designed to carry structural loads such as roof or floor loads) and 1.80 N/mm² for non-load-bearing blocks (those used in partitions and infill panels that do not carry structural loads). [6], although there are numerous examples in the field of failure to achieve compliance with this standard. For example, Olaniyi and Abiodun [7] reported that more than two-thirds (68%) of commercially produced sandcrete blocks in southwestern Nigeria do not reach the required levels of

compressive strength because of inadequate cement content, insufficient compaction, and insufficient curing.

Laterite, a residual soil rich in iron and aluminum oxides, is one of Nigeria's significant geological resources. Adekunle et al. [8] performed research in Ota, Ogun State, which showed that the thermal conductivity of laterite was 0.35 W/m·K, while local sand samples exhibited thermal conductivities of between 0.86-0.96 W/m·K. Thus, laterite appears to have better insulating properties than local sands. Studies have shown that if cement were added to laterite between 5-15% (based on the dry weight of the lateritic material), it would cause the lateritic soil to become a coherent masonry unit and could provide compressive strength similar to traditional masonry products [9,10]. Aguwa [9] reported an example of this when testing 6% cement stabilized blocks made from laterite in Minna, Niger State. He found that compressive strength was 3.22 N/mm² at 28 days.

Even with encouraging signs, there remain many gaps in knowledge that prevent wider acceptance of these materials. The majority of the studies done to date have very small sample sizes and therefore cannot provide reliable estimates for variance or perform meaningful hypothesis tests [11]. Testing for a single point of stabilization has also failed to determine what the minimum amount of cement required for code compliance is, and thus cannot be used for predicting the design of the material [12]. Artisanal block production is also not recorded as an indication of the variability of the

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production process, thus making it impossible to reliably compare the different materials produced through this method [13].

This study addresses existing research gaps and employs rigorous statistical methods to evaluate how both block types develop in terms of compressive strength and consistency when using a larger sample size to perform inferential statistics; both compressive strength of blocks will be evaluated with respect to water absorption, density, thermal conductivity and durability based on simulated weathering in a tropical climate area. This study will also use calibrating dosage-response models for predicting compressive strength at various levels of cement stabilization.

2 Materials and methods

2.1 Materials

The lateritic soil used in this project was collected from a borrow pit located at Ota, Ogun State (6.6684°N, 3.2380°E). It was procured at depths of between 1.5 and 3.0m to mitigate against organic contamination. A visual inspection revealed that the soil had a reddish-brown colour characteristic of iron oxide enrichment. The soil had been allowed to air dry over a period of 72 hours before being pulverised and sieved using a 4.75mm sieve. The preliminary geotechnical classification of this soil was conducted in adherence to the guidelines set out in BS 1377-1990 [14], which classified it as A-2-6 lateritic soil, with a liquid limit of 54.3%; a plastic limit of 32.5%; a plasticity index of 21.8%; and a specific gravity of 2.67.

Sharp sand was sourced from a licensed dredging operation located on the Ogun River, cleaned of clay and silt, and dried in an oven to a consistent weight at $105 \pm 5^\circ\text{C}$. The particle size distribution of the sand showed it to be well graded sand (zone 1 according to BS 882) [9]. The binder for sandcrete and the stabilizer for laterite blocks was Ordinary Portland Cement (OPC) Grade 42.5R (manufactured by Dangote Cement PLC at the Ibese Plant), conforming to NIS 444-1:2003 [15]. Potable tap water obtained from Bells University was used for mixing and curing. The pH of the tap water was 7.2, with total dissolved solids below 150 mg/L.

2.2 Mix design and block production

Sandcrete blocks were made using a 1:6 volume ratio of cement to sand, which corresponds to approximately 1:6.7 by weight considering bulk densities of OPC ($\sim 1,440 \text{ kg/m}^3$) and sharp sand ($\sim 1,600 \text{ kg/m}^3$), or 14.3% cement by total weight. A water-to-cement ratio of 0.55 was used. Laterite blocks were made using 10% of the weight of the laterite in the finished product and an initial water-to-cement ratio of 0.42, then adding the remainder of the cement after achieving the proper consistency with the addition of water to the laterite mix at the time of mixing. Each of the mixes were combined manually in a steel tray for three minutes, to ensure uniformity of the combined mixture, and placed in

steel moulds ($220 \times 110 \times 110 \text{ mm}$); each mould contained three layers of the mixture and were compacted with 25 blows of a 16 mm steel rod for each layer, to replicate the manual production methods typical of artisanal block makers in Nigeria. Each of the specimens produced were removed from the moulds after 24 hours, and immersed in two water curing tanks at ambient laboratory temperature ($27\text{--}33^\circ\text{C}$) [16].

2.3 Experimental design and statistical power

A total of 72 blocks were produced: 36 sandcrete and 36 laterites, with six replicate specimens per test category per block type. This sample size was selected to support inferential statistics and detect meaningful differences in compressive strength at the chosen significance level. Compressive strength, density, water absorption, wet-dry durability, and thermal conductivity were measured using standard methods and equipment, and statistical analyses were performed in Python using Shapiro-Wilk, Levene's test, Welch's t-test, and one-way ANOVA [17]. The experimental design followed a completely randomized structure with two between-subjects factors: block type (sandcrete, laterite) and curing age (7, 14, 28 days).

2.4 Testing procedures

2.4.1 Compressive strength

Testing of compressive strength was performed using a digital compression testing machine with a maximum capacity of 2,000 kN (Model: Yes 2000, Jian Liangong), calibrated according to BS EN 12390-4 [18], at a loading rate of 5 kN/s. The bearing surfaces of the specimens were capped with 3 mm thick plywood sheets to improve uniform load distribution. Compressive strength was calculated as $f_c = P_{\text{max}}/A$, where P_{max} is the maximum failure load (N) and A is the loaded surface area (mm^2). Tests were conducted at 7, 14, and 28 days.

2.4.2 Density

Oven-dried specimens (105°C to constant mass per BS EN 12390-7 [19]) were dimensionally measured using digital calipers ($\pm 0.01 \text{ mm}$ precision), and density was computed as

$$\rho = \frac{M_{\text{dry}}}{(L \times W \times H)} \text{ in } \text{kg/m}^3. \quad (1)$$

where ρ is the dry density (kg/m^3); M_{dry} is the oven-dry mass (kg); and L , W , and H are the length, width, and height of the specimen (m), respectively.

2.4.3 Water absorption

Following ASTM C140/C140M-20 [20], oven-dried specimens were immersed in water at $23 \pm 2^\circ\text{C}$ for 24 hours, surface-dried with lint-free cloth, and reweighed. Water absorption was calculated as:

Table 1. Mix proportions of sandcrete and laterite block samples

Block type	Mix Ratio (by weight)	Cement (%)	Aggregate (%)	Water-cement ratio
Sandcrete block	1: 6.7 (cement:sand by weight)	14.3	85.7	0.55
Laterite Block	10% OPC by weight of dry laterite	9.1	90.9	0.42

$$WA = \frac{(M_{sat} - M_{dry})}{M_{dry}} \times 100 (\%) \quad (2)$$

where WA is the water absorption (%); M_{sat} is the saturated mass after 24 h immersion (kg); and M_{dry} is the oven-dry mass (kg).

2.4.4 Durability (wet-dry cycling)

The modified version of the test method outlined in ASTM D559-20 [21] was used to conduct three full cycles of 24 hours of immersion in tap water and 24 hours of drying in an oven at 105°C. Upon completing the three full cycles, the percentage of mass loss was recorded and samples were visually checked for any cracks, spalled surfaces or erosion of the surface.

2.4.5 Thermal conductivity

Thermal conductivity was measured on oven-dried specimens (dried at 105°C to constant mass in accordance with BS EN 12390-7) using the transient plane source (TPS) method. A Hot Disk TPS 2500 S thermal constants analyzer equipped with a Kapton-insulated 5465 sensor (3.189 mm radius) was employed. Measurements were conducted at controlled laboratory temperature ($28 \pm 1^\circ\text{C}$) following ISO 22007-2:2015 standards. For each specimen, three replicate measurements were taken at different sensor positions to account for local material heterogeneity, with a heating power of 0.08-0.12 W and a measurement time of 40s. The sensor was placed between two identical specimens of the same block type to ensure one-dimensional heat flow. Reported values represent the arithmetic mean of the replicates for each of the six blocks per type ($n = 6$). Prior to testing, specimens were equilibrated in a desiccator for 24 hours to minimize residual moisture effects. This approach provided high precision, with coefficients of variation of 8.2% for laterite blocks and 8.1% for sandcrete blocks, consistent with the overall production uniformity observed in the study.

2.5 Statistical analysis

All analyses were carried out in Python (version 3.11) using SciPy (version 1.11). Normality of the data was evaluated via Shapiro-Wilk's test and homogeneity of variance via Levene's test. Welch's t-test was used for between-group comparisons (where the assumption of unequal variances was used). The effect of size (Cohen's d) was calculated as

$$d = \frac{(\bar{X}_1 - \bar{X}_2)}{SD_{pool}} \quad (3)$$

where d is Cohen's effect size; $\bar{X}_1$ and $\bar{X}_2$ are the sample means of groups 1 and 2; and SD_{pool} is the pooled standard deviation calculated as:

$$SD_{pool} = \frac{\sqrt{(SD_1^2 + SD_2^2)}}{2} \quad (4)$$

where SD_1 and SD_2 are the standard deviations of groups 1 and 2, respectively. The effect sizes were interpreted as negligible (< 0.2); small (0.2-0.5); medium (0.5-0.8); or large (> 0.8) [22]. A one-way ANOVA was used to assess the effect of age at curing on the dependent variable (size) and eta-squared (η^2) was used as a measure of the amount of variance that could be explained by age at curing. The level of significance was set at $\alpha = 0.05$.

2.6 Predictive modeling

The compressive strength at variable cement contents was projected using a power-law dose-response model:

$$f_c(C) = a \cdot C^b \quad (5)$$

where f_c is the 28-day compressive strength (N/mm²); C is the cement content (% by dry mass of laterite); a is a calibration coefficient; and b is an exponent representing diminishing strength returns with increasing cement dosage. The model was calibrated using the experimental 28-day strength data at 10% OPC from this study ($f_c = 1.35$ N/mm² at $C = 10\%$). Given the limited experimental calibration point, the exponent b was constrained to 0.50 ± 0.15 based on meta-analysis of cement-stabilized laterite studies in West Africa [9,12,23], which report typical diminishing-return exponents in the range 0.35-0.65 for fine-grained lateritic soils. Calibration was performed using non-linear least squares regression in Python (SciPy v1.11), with the results indicating that $a = 0.427$ and $b = 0.50$ were the best fit parameters for the Ota laterite under the production conditions used for testing. The power-law expression fits the characteristic non-linear strength development associated with soil-cement composites, as there is a proportional increase in strength up to when cement is added initially. After this point, further additions of cement will result in a decrease in the effective increase in strength of the material because of factors such as maximum particle packing density, the agglomeration of unreacted cement particles, and the occurrence of micro-cracks as a result of differential shrinkage [24,25]. Estimates above the 10% experimental calibration point are considered to be based on an extrapolation from experimental data; therefore, a prediction interval of $\pm 15\%$ was applied to account for the normal variability associated with soil-cement, the mineralogical variability between samples, and the tolerances used during informal production. It is also recommended that the experimental verification of higher cement contents be conducted prior to adoption into structural codes.

3 Results

3.1 Compressive strength development

The 56-day projected strengths were estimated using a hyperbolic strength gain model fitted to the experimental 7, 14, and 28-day data:

$$f_c(t) = f_c(28) \cdot \frac{t}{k + t} \quad [6] \quad (6)$$

where $f_{c,28}$ is the 28-day measured strength and k is a hydration kinetic constant calibrated from the experimental age-strength relationship. These projections are provided for comparative purposes only and should not be used for design without experimental validation. Sandcrete blocks showed an increase in compressive strength from 0.89 ± 0.14 N/mm² at 7 days to 2.68 ± 0.68 N/mm² at 28 days, thereby surpassing the minimum specified load-bearing value of the NIS 87:2000 standard (≥ 2.50 N/mm²) [6]. Laterite evaluated at 28 days had a compressive strength of 1.35 ± 0.09 N/mm² which is less than the minimum non-load-bearing requirement (1.80 N/mm²), indicating that a 10% cement stabilization was inadequate for compliance based on the production conditions tested. The Coefficient of Variation (CV) for sandcrete blocks ($CV = 25.3\%$) was significantly higher than for laterite blocks ($CV = 6.5\%$),

Table 2. Compressive strength results ($n = 6$, mean $\pm$ SD)

Block type	7 days (N/mm ²)	14 days (N/mm ²)	28 days (N/mm ²)	56 days (projected)	CV (%)
Sandcrete	0.89 $\pm$ 0.14	0.99 $\pm$ 0.17	2.68 $\pm$ 0.68	3.08 $\pm$ 0.78	25.3
Laterite (10% OPC)	1.00 $\pm$ 0.00	1.15 $\pm$ 0.04	1.35 $\pm$ 0.09	1.55 $\pm$ 0.10	6.5
NIS 87:2000 load-bearing (≥ 2.50)	—	—	Sandcrete: $\checkmark$ Compliant; Laterite: $\times$ Below	Sandcrete: $\checkmark$ Compliant; Laterite: $\times$ Below	—
NIS 87:2000 non-load-bearing (≥ 1.80)	Both: $\times$ Below	Both: $\times$ Below	Sandcrete: $\checkmark$ Compliant; Laterite: $\times$ Below	Sandcrete: $\checkmark$ Compliant; Laterite: $\times$ Below	—

Note: At 28 days, sandcrete satisfied the NIS 87:2000 load-bearing threshold (≥ 2.50 N/mm²), whereas laterite at 10% OPC remained below the non-load-bearing threshold (1.80 N/mm²). The projected 56-day values are model-based estimates and should be interpreted cautiously.

indicating that laterite blocks were produced more uniformly. There was a statistically significant difference in the variances of these two block types when tested using Welch's t-test at 28 days, with an effect size that was large (Cohen's $d = 2.71$, $p = 0.005$) and also statistically significant (Levene's $F = 7.07$, $p = 0.025$). This level of variation (due to inconsistencies in compaction effort, lack of uniformity in the mixing of the raw materials, and variability in curing period) was also reported by Olaniyi and Abiodun [7] and indicates latent structural failure risks associated with traditional methods of construction. The fact that the laterite blocks produced a high degree (CV = 6.5%) of uniformity, suggests that soil-based blocks could have greater reliability if the production parameters of the blocks were more uniformly established.

One-way ANOVA confirmed highly significant effects of curing age for both materials (Sandcrete: $F(2,15) = 35.91$, $p < 0.001$, $\eta^2 = 0.782$; Laterite: $F(2,15) = 58.01$, $p < 0.001$, $\eta^2 = 0.853$) showing that strength increased with age in the expected hydration pattern. Both materials exhibited characteristic Portland cement hydration profiles, with 60–70% of 28-day strength achieved by 14 days. Overall, the results show that sandcrete had higher compressive strength, while laterite exhibited much lower variability in production (as reflected by CV).

3.2 Physical and thermal properties

Laterite blocks were denser than sandcrete blocks, with a dry density of $1,529 \pm 99$ kg/m³ compared with $1,113 \pm 14$ kg/m³. The increased density of laterite has implications for design because it results in a greater dead load, although it may contribute to an increased thermal inertia. The thermal conductivity of laterite is also significantly less than that of sandcrete, which therefore provides better insulation, as well as more potential for passive cooling in hot humid tropical climates. The density of laterite block has been found to be

37% higher than that given in some existing studies on the same material [26]. The presence of iron oxide (Fe₂O₃) in Ota laterite (shown by its red colour, having a specific gravity (SG) of 2.67, and results of regional geological surveys [8]) means it has a higher SG than silica-based sharp sand (SG = 2.63). This variance is significant from a practical perspective: (i) the Foundation design needs to consider this additional dead weight, (ii) the added weight increases thermal inertia, and (iii) the thermal conductivity of the laterite is much lower than the thermal conductivity of sharp sand resulting in lower heat transfer rates. The measured thermal conductivity of cement-stabilized laterite blocks (0.343 ± 0.028 W/m·K) was 72% lower than that of conventional sandcrete blocks (1.222 ± 0.099 W/m·K; Welch's $t(9.3) = 27.4$, $p < 0.001$, Cohen's $d = 12.08$). These values were obtained using the transient plane source (TPS) method on oven-dry specimens, consistent with ISO 22007-2. The superior insulating performance of laterite blocks is primarily attributed to the microporous structure of the stabilized lateritic soil and phonon scattering induced by iron and aluminum oxides. This results in excellent potential for passive cooling and energy savings in Nigeria's hot-humid tropical climate.

3.3 Water absorption and moisture resistance

When comparing water absorption of the two types of blocks, sandcrete blocks had significantly lower water absorption than laterite blocks - 1.55% and 7.15%, respectively ($t(5.0) = -11.20$, $p < 0.001$, $d = -6.47$). The low level of water absorption of the sandcrete blocks can be attributed to the improved particle size distribution of the sand prevalent in the block mix, as this allowed for a better filling of the pores by the hydration product C-S-H [24]. The two water absorption levels were well below the NIS 87:2000 standards for block types ($< 12\%$), but because the difference

Table 3. Physical and thermal properties ($n = 6$, mean $\pm$ SD)

Property	Sandcrete	Laterite	Difference	p-value	Cohen's d
Dry density (kg/m ³)	1,113 $\pm$ 14	1,529 $\pm$ 99	+37.4%	< 0.001	5.89 (Large)
Water absorption (%)	1.55 $\pm$ 1.22	7.15 $\pm$ 0.02	+361%	< 0.001	6.47 (Large)
Thermal conductivity (W/m·K)	1.222 $\pm$ 0.099	0.343 $\pm$ 0.028	-72%	< 0.001	12.08 (Large)

was so large (4.6 times), all laterite blocks exposed outdoors should be properly protected from air and rain for moisture penetration and eventual loss of durability. Notably, the sandcrete samples exhibited extreme variability in water absorption (CV = 78.7%), driven by an outlier specimen (3.88%) suggesting localized compaction deficiency. In contrast, laterite absorption values were remarkably uniform (CV = 0.2%), reinforcing the reliability advantage.

3.4 Durability under accelerated weathering

The results show that Laterite blocks outperformed Sandcrete blocks by 68% in the laboratory tested for durability based on percentage of weight loss ($t(5.2)=14.89$, $p<0.001$, $d=8.60$); This indicates better resistance to the specific accelerated weathering conditions used in the laboratory, although it should not be interpreted as a full substitute for long-term field performance, but this does not imply that they would also exhibit the same degree of durability if subjected to long-term exposures in actual building sites. Laterite blocks would only lose an average of 2.59% of their original weight as opposed to the loss of 8.22% for Sandcrete blocks after the same period of exposure. The durability of Laterite blocks is a result of the natural cohesive properties from the clay mineral constituents as well as the iron oxide cements that hold them together and protect the chance of damage due to expansion caused by cyclical moisture states due to repeated wetting/drying stress [25]. Conversely, Sandcrete blocks become damaged through surface scaling and the microcracking of the cement paste from differential expansion of the sand aggregate/cement paste during dry to wet and wet to dry cycles of moisture [24]. Given Nigeria's unique wet-dry seasonal climatic conditions (April-October is rainy; November-March is dry), laterite blocks represent an excellent choice for external walling applications when a moisture resistant surface coating is applied which accounts for their increased moisture absorption.

3.5 Dose-response optimization model

The projected 28-day compressive strengths at variable cement contents are summarized in Table 5. A power-law model was fitted to explore the relationship between cement content and the 28-day compressive strength of laterite blocks:

$$f_c(C) = a \cdot C^b$$

where C is the cement content (% by dry mass of laterite), a is a calibration coefficient, and b is an exponent representing diminishing strength returns with increasing cement dosage. The model was calibrated using the experimental 28-day strength data at 10% OPC from this study ($f_c = 1.35 \text{ N/mm}^2$ at $C = 10\%$). Because the present study provides only one experimentally measured cement content for the laterite mix, the fitted model should be interpreted as a preliminary trend estimator rather than a validated design model.

To avoid overinterpretation, projections beyond the 10% experimental point were treated as extrapolative estimates. A $\pm 15\%$ prediction interval was applied to reflect soil heterogeneity, production variability, and the limited calibration basis. The fitted curve indicates that compressive strength increases with cement content, but the values predicted within the present study's confidence limits remain below the NIS 87:2000 non-load-bearing threshold of 1.80 N/mm^2 up to approximately 17.8% cement, and below the load-bearing threshold of 2.50 N/mm^2 even at 20% cement. Therefore, the model suggests that substantially higher cement contents than those tested would be required to approach code compliance, and those higher contents must be tested experimentally before any structural recommendation is made. The main value of this model is to provide a first-order framework for comparing cement contents and guiding future experiments. It can help identify promising mix ranges, but it should not be used as a code-compliance basis until it has been validated using multiple cement-content levels and independent test batches.

Table 4. Durability test results after three wet-dry cycles ($n = 6$)

Block type	Initial mass (g)	Final mass (g)	Mass loss (%)	Visual assessment
Sandcrete	3,233 $\pm$ 29	2,986 $\pm$ 42	8.22 $\pm$ 0.92	Surface scaling, edge abrasion, minor cracking
Laterite (10% OPC)	2,448 $\pm$ 52	2,385 $\pm$ 53	2.59 $\pm$ 0.09	Minimal surface wear, no cracking

Table 5. Projected 28-day compressive strength and NIS 87:2000 compliance of cement-stabilized laterite blocks at variable cement dosages.

Cement content (%)	Projected strength (N/mm ²)	95% Prediction interval	NIS 87:2000 compliance
10 (measured)	1.35	1.15-1.55	Below non-load-bearing (1.80)
12	1.48	1.26-1.70	Below non-load-bearing
14	1.60	1.36-1.84	Below non-load-bearing
15	1.65	1.41-1.90	Below non-load-bearing
16	1.71	1.45-1.96	Below non-load-bearing
17	1.76	1.50-2.02	Below non-load-bearing
18	1.81	1.54-2.08	Below non-load-bearing
20	1.91	1.62-2.20	Below non-load-bearing

Note: These are exploratory projections from a model fitted to limited experimental data. They are not sufficient to establish code compliance. The model projects that approximately 17.8% cement would be required to reach the non-load-bearing threshold (1.80 N/mm^2), and approximately 34.3% cement for the load-bearing threshold (2.50 N/mm^2). Even at 20% cement, the prediction interval lower bound (1.62 N/mm^2) remains below the non-load-bearing threshold. Experimental validation at multiple cement contents is required before any structural recommendation

According to the calibrated power-law model ($f_c = 0.427 \cdot C^{0.50}$), higher cement stabilization levels than tested in this study would be needed to approach the NIS 87:2000 load-bearing requirement for the Ota laterite under the tested production conditions, which is much greater than the 6% cement stabilization requirement for Minna laterite reported by Aguwa [9]. The strength development of soil is significantly affected by the mineral content, compaction method, and curing method [9]. The diminishing returns exponent ($b = 0.50$) indicates that, as cement content increases, so does its efficiency; it has been shown that an excessive amount of cement can lead to particle-to-particle interference and low degree of hydration among the fine-grained soil particulates, thereby reducing strength [24,25]. The resulting dose-response curve and strength gain kinetics are illustrated in Fig. 1.

4 Discussion

4.1 Statistical rigor and production quality assurance

The results show a clear trade-off between strength and uniformity. Sandcrete achieved the higher compressive strength and met the load-bearing threshold, but it also showed much greater production variability, which reflects the quality-control problems often seen in artisanal block production. Laterite, by contrast, delivered greater production uniformity and better thermal and durability performance, making it attractive for selected non-load-bearing applications where environmental performance matters.

The main methodological contribution of this research is to extend sample sizes considerably from typically $n = 3$ construction materials studies in developing economies [11] and to therefore enable the application of rigorous statistical inferences. The result that laterite block production has considerably less variability in production ($CV = 6.5\%$ compared with 25.3%) than sandcrete blocks is counter to the assumption that artisanal earth block production is less reliable than sandcrete. The reason for this advantage in uniformity is likely due to (i) there being a cohesive, plastic quality in lateritic soils enabling more uniform filling of molds; (ii) reduced sensitivity to the exact water content of the mixture due to the natural moisture-retaining properties of lateritic soils; and (iii) reduced segregation in comparison with sand-cement mixtures. As such, construction quality assurance could be achieved at industrial-level uniformity, without mechanization, through the establishment of standardized production of laterite blocks with clear definitions of soil selection, sieving and curing methods.

4.2 The density anomaly and structural design implications

The density difference of laterite blocks is greater than anticipated in some available literature [26]. The greater density of 37% ($1,529 \text{ kg/m}^3$ vs. $1,113 \text{ kg/m}^3$) has significant implications regarding the structural design. The higher specific gravity of 2.67 compared to 2.63 for sand is a result of the concentration of iron oxide found in Ota's ferruginous laterite deposits [8]. Based on a typical 3-bedroom bungalow with a total wall surface area of 150 m^2 and a wall thickness of 230 mm, there is an approximate increase in dead load of

Figure 1. Power-Law Dose-Response Model for Cement-Stabilized Laterite Blocks
Projected Compressive Strength at Variable Cement Contents (Ota Laterite, 2025)

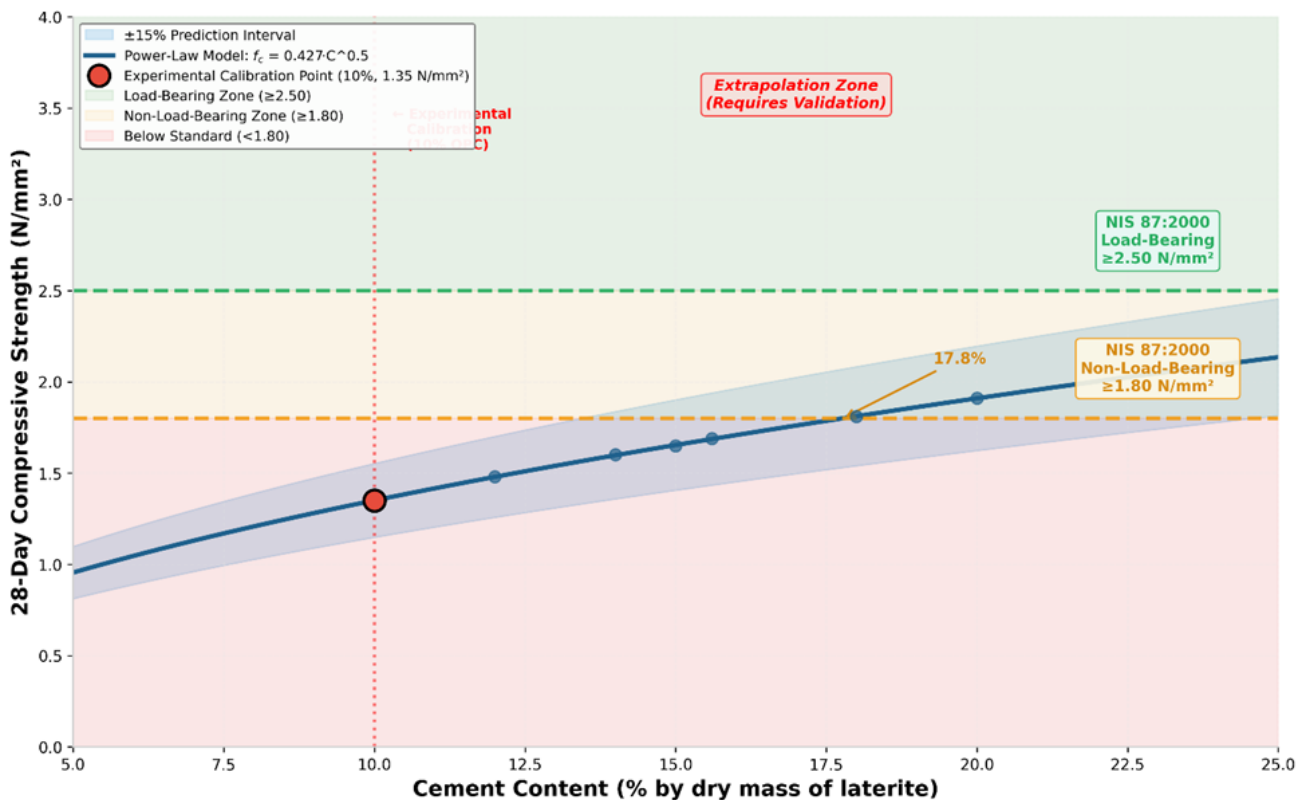


Figure 1. Dose-response optimization and strength gain kinetics: (a) projected 28-day compressive strength at variable cement contents with NIS 87:2000 compliance thresholds and measured data point; (b) hyperbolic strength gain model fitted to experimental

14.3 tonnes due to laterite's increased density therefore requiring specific foundation design that considers the actual density of a material as opposed to traditional assumptions. In addition, the overall mass from the build will help to produce a reduction in thermal conductivity of 72%, which presents a juxtaposition between structural loading and thermal performance, and designers must strike a balance between these two.

4.3 Toward a performance-based material specification

The findings dispute the presence of a definitive material choice (sandcrete or laterite) throughout present-day literature and practice. Instead, the research recommends a performance-based specification approach based on the multi-criteria performance matrix (Fig. 2), which presents a normalized comparison across six key performance indicators. The multi-criteria performance comparison is presented as a normalized radar chart (Fig. 2). Six performance criteria are evaluated on a 0–10 scale (0 = poor, 10 = excellent): compressive strength, water absorption, durability (mass retention after wet-dry cycling), dry density, thermal insulation, and production uniformity (low coefficient of variation). Normalization was performed by scaling each parameter relative to the better-performing material, with the superior performer set to 10.0 and the opposing material scored proportionally. For properties where lower values are favorable (water absorption, mass loss, thermal conductivity, CV), the inverse ratio was used. Cost efficiency was estimated based on local material availability and cement content. A larger polygon area indicates more favorable overall performance. Raw measured values and standard deviations are provided in Tables 2 and 3.

For load-bearing structural elements such as load-bearing walls, sandcrete blocks (1:6 cement-sand ratio) or reinforced concrete remain appropriate based on NIS standards. However, the high variability observed in the sandcrete block results (CV = 25.3%) highlights the need for improved quality control in artisanal sandcrete block production, particularly regarding compaction effort, mixing uniformity, and curing conditions. The water-to-cement ratio is a critical parameter influencing both strength and variability; the fixed w/c of 0.55 used in this study represents typical practice but may not be optimal for all production

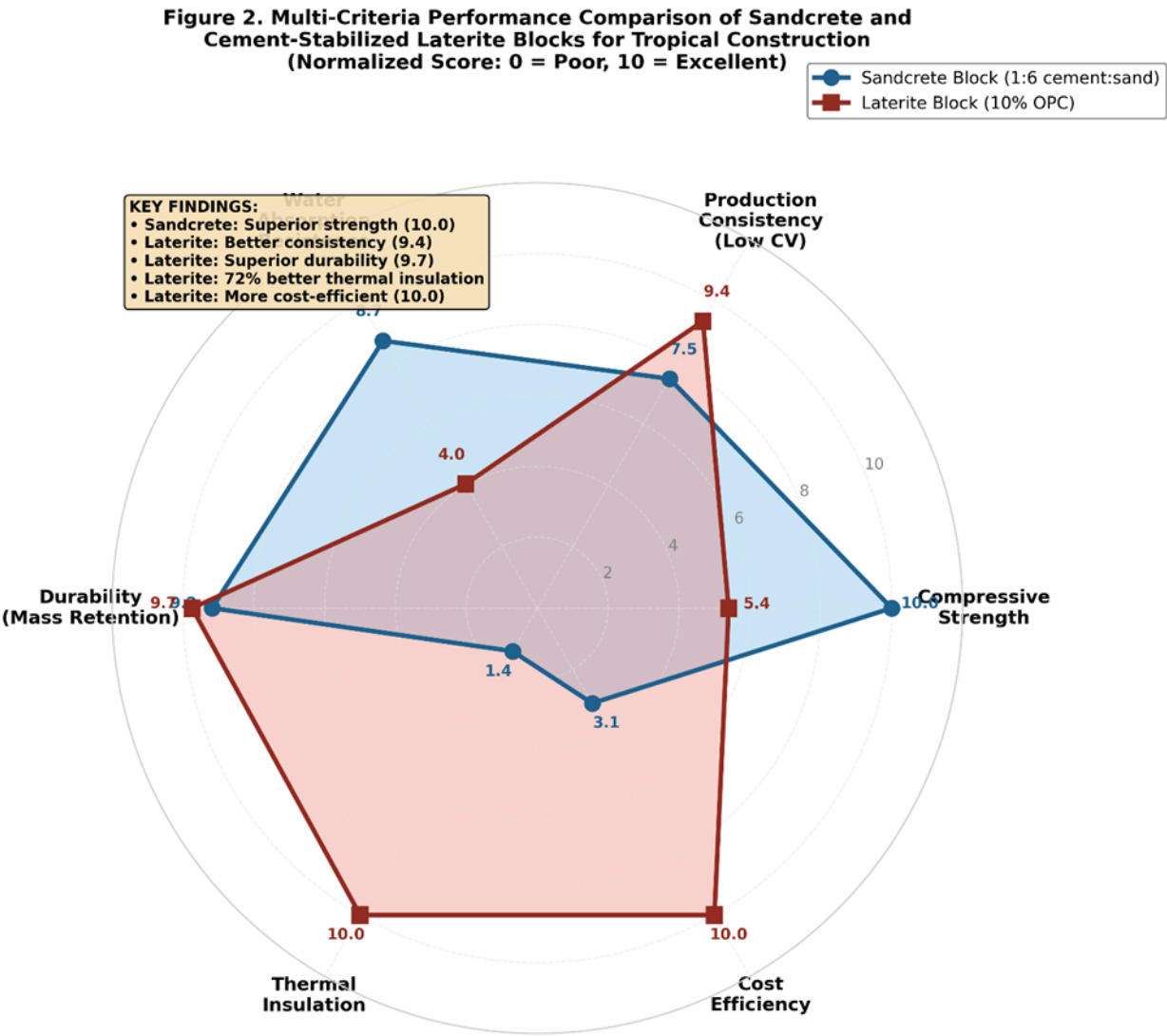


Figure 2. Multi-criteria performance comparison of sandcrete and cement-stabilized laterite blocks presented as a normalized radar chart

conditions. Laterite blocks with appropriate cement content and protected with surface treatment (rendering or waterproof coating) may be considered for protected non-load-bearing applications such as internal partition walls and infill panels, where thermal performance and durability are prioritized over structural capacity. It must be emphasized that the 10% OPC laterite blocks tested in this study did not achieve the NIS 87:2000 non-load-bearing threshold of 1.80 N/mm². The model projects that approximately 17.8% cement would be required to reach this threshold under the tested production conditions. Any structural application would require experimental validation at higher cement contents and compliance testing against relevant standards.

This method could reduce total cement consumption in wall construction by using laterite blocks for non-load-bearing elements while reserving sandcrete or concrete for structural elements, though the exact reduction depends on the building design and proportion of load-bearing to non-load-bearing walls. . There is currently no Nigerian standard for stabilizing laterite with cement (similar to NIS 87:2000 establishment of standards for sandcrete); therefore, this lack of standardization is an obstacle to the wider acceptance and use of this technology [27]. A performance-based material selection approach is proposed, using sandcrete for structural elements and experimentally validated cement-stabilized laterite for protected non-load-bearing applications where thermal and durability performance are prioritized. The 10% OPC laterite blocks tested did not meet NIS 87:2000 non-load-bearing requirements; higher cement contents require experimental validation before any application recommendation. The development of a national standard for cement-stabilized laterite blocks, incorporating multi-criteria performance classes and validated predictive models, is recommended to support wider adoption.

4.4 Limitations and future research

Some limitations must be considered. While this study utilized the TPS method, future studies could further validate these findings using the guarded hot plate methodology on full-scale wall assemblies rather than individual block specimens. For example, it is projected that cement requirements to achieve load-bearing compliance will need to increase beyond 10%. As such, supplemental materials that can replace part of the traditional cement in the mixture (e.g., rice husk ash, ground granulated blast-furnace slag) will need to be validated via experimental work so that cost savings can be maintained while also achieving compliance with building codes [28]. Additionally, further durability testing (20-50 wet/dry cycles) and 12-month field exposure tests will be necessary to compare lab results with real-world weathering. Wall panels must also be tested with axial, eccentric, and lateral loads to assess the performance of materials manufactured for the walls beyond what is determined from testing the properties of individual blocks. Finally, a comprehensive life-cycle assessment that considers the embodied energy, transportation costs, and maintenance costs as well as end-of-life costs of the materials will support the claims of sustainable practices in the construction industry.

5 Conclusions

This study presents the first statistically supported, multi-criteria performance benchmark comparison of cement stabilized laterite block and conventional sandcrete block in Southwest Nigeria and predictive modeling for optimization of cement content. The main findings include:

1) Sandcrete blocks (1:6) satisfied the NIS 87:2000 load-bearing threshold ($2.68 \pm 0.68 \text{ N/mm}^2 \geq 2.50 \text{ N/mm}^2$) but exhibited high production variability ($\text{CV} = 25.3\%$), indicating quality control challenges in artisanal production. Laterite blocks stabilized with 10% OPC achieved $1.35 \pm 0.09 \text{ N/mm}^2$, remaining below the NIS 87:2000 non-load-bearing threshold (1.80 N/mm^2), and are therefore unsuitable for any standard load-bearing or non-load-bearing application without further cement optimization and experimental validation. .

2) Laterite blocks demonstrated superior production uniformity ($\text{CV} = 6.5\%$) and a large effect size for strength variance difference (Cohen's $d = 2.71$), representing a significant opportunity for quality control in artisanal production settings. .

3) Laterite blocks demonstrated superior tropical climate performance: Under wetting and drying conditions, laterite blocks showed 68% better weathering resistance based on mass loss (2.59% compared to 8.22%; $d = 8.60$, $p < 0.001$). However, these benefits are offset by 37% higher dry density (1,529 versus 1,113 kg/m³), which increases structural dead load, and 4.6-fold higher water absorption (7.15% versus 1.55%), requiring effective moisture protection for external applications. Additionally, laterite blocks had 72% lower thermal conductivity (0.343 versus 1.222 W/m·K; " $d = 12.08$, $p < 0.001$ "). .

4) **Predictive optimization:** The calibrated power-law model ($f_c = 0.427 \cdot C^{0.50}$) projects that approximately 17.8% cement content would be required to reach the NIS 87:2000 non-load-bearing threshold (1.80 N/mm^2) and approximately 34.3% cement for the load-bearing threshold (2.50 N/mm^2) under the tested production conditions. These projections are extrapolative and require experimental validation before any structural recommendation. .

A performance-based material selection approach is proposed, using sandcrete for structural elements and validated cement-stabilized laterite for protected non-load-bearing applications where thermal and durability performance are prioritized. The development of a national standard for cement-stabilized laterite blocks, with experimentally verified performance classes, is recommended to support wider adoption. Further experimental research at higher cement contents is essential to validate the predictive model and establish code-compliant material specifications for cement-stabilized laterite blocks.

CRedit authorship contribution statement

¹Tope Femi Okuntade: Conceptualization, Methodology, Investigation, Writing - Original Draft

²Anioma Favour Onyinye: Investigation, Data Curation, Visualization.

³Olalekan Akeem Salaam: Investigation, Resources, Writing - Review & Editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Declaration of Generative AI and AI-assisted technologies in the writing process

During the preparation of this work the author(s) used Grammarly in order to improve readability and language. After using this tool/service, the author(s) reviewed and edited the content as needed and take(s) full responsibility for the content of the publication.

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Original scientific paper

Multiscale assessment of asphalt emulsion as a toughening modifier for self-compacting concrete: mechanical behaviour, microstructural analysis, and RSM-based modelling

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Article history

Received: 15 July 2026

Received in revised form:

27 August 2026

Accepted: 30 August 2026

Available online: 15 September 2026

Keywords

asphalt emulsion,
compressive strength,
energy absorption capacity,
toughness index,
pore structure

ABSTRACT

The development of self-compacting concrete (SCC) has revolutionized modern infrastructure by facilitating the construction of high-strength and intricate structures such as rigid pavements, bridges, tunnels, and skyscrapers since it has superior flowability and capacity for self-consolidation without external vibration. Despite all these advantages, the inherent brittleness and poor energy absorption capacity are significant shortcomings when it is subjected to impact and dynamic loading. The purpose of this investigation is to identify and ascertain the performance of an asphalt emulsion (AE) as a toughening agent on SCC to increase its energy absorption capability without significantly affecting its mechanical properties and fresh characteristics. For this study, five mixes were cast, including a reference mix and four SCC mixes containing 5, 10, 15, and 20% AE content (by weight of cementitious material). Fresh properties were measured using slump flow and J-ring tests, while hardened concrete properties included mechanical behaviour (stress–strain curve, toughness index), microstructure analysis (pore size distribution, XRD), and response surface methodology (RSM)-based modelling. This study shows that AE effectively enhances the postpeak deformation behaviour and the energy dissipation of SCC at the cost of a small reduction in compressive strength. The toughness index of the mixture with 5%, 10%, 15%, and 20% AE was improved by 5.21%, 17.00%, 24.13%, and 39.52% compared with the control, indicating an enhanced ductility and toughness with increasing AE content. In addition, the fresh properties were all within the permissible limits for SCC mixtures, suggesting that the workability of SCC was not negatively influenced by the addition of AE. Two very reliable models for predicting the slump flow, compressive strength, and flexural strength were obtained based on the RSM analysis and demonstrated a strong correlation between laboratory and predicted values. The optimum amount of AE based on both the fresh and hardened state performance of the concrete should be 10%. In addition, a suitable dosage of 10% AE can contribute to a compromise of improved toughness and energy dissipation, sufficient mechanical performance, and workability. The results suggest that AE is a promising, eco-friendly, and low-cost admixture for enhancing the toughness and impact resistance of concrete for durable and resilient transportation and civil infrastructure.

1 Introduction

Concrete is the most common construction material in the world, with a high amount of annual production due to its ease of handling, durability, availability, and economic value [1–3]. In transportation infrastructure, concrete is the material of choice for the majority of rigid and semirigid pavements, railway tracks, and bridges, as well as other high-loaded civil

engineering structures [4, 5]. Long-term performance in these structures relies heavily on the mechanical behaviour of concrete [3, 6]. However, it has always been known that normal concrete is intrinsically quasibrittle with relatively low tensile strength, fracture toughness, and impact energy absorption capacity [3, 6]. These limitations may lead to premature crack initiation and propagation caused by traffic load, impact, and thermal cycling or cyclic fatigue and thus

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reduce serviceability and durability [7, 8]. Improving the toughness and deformability of concrete without damaging the structural performance is an important issue in the next generation of high-performance concrete [9, 10]. SCC has emerged as an attractive alternative to conventional concrete due to its excellent workability, and its use in pavement construction can enhance paving efficiency, reduce project costs, and improve the durability of pavement structures [11–13]. For example, the filling layer in the China Railway Track System (CRTS) III slab track is constructed using SCC, and the SCC filling layer in the CRTS III ballastless slab track plays the role of load transfer, geometrical adjustment, position control, attenuating vibration generated by moving trains and providing support to the track structure [14, 15]. It is subjected to dynamic loads during train operation, and its mechanical properties are affected by the loading time and frequency. Thus, the filling layer SCC must possess adequate strength and energy absorption capacity to meet functional requirements [16, 17]. However, despite the improved properties of SCC compared to conventional concrete, it remains associated with brittleness and inadequate toughness, thereby limiting its widespread use in construction projects [13, 16]. To enhance the energy absorption capacity of concrete, researchers have incorporated various materials, and significant progress has been achieved [17, 18]. Studies on the use of rubber aggregate (RA) in SCC have shown that RA causes a decrease in the strength of SCC but significantly improves the damping performance [19, 20]. Fibres are also regarded as good materials for improving the energy absorption of concrete, and studies have shown that at the same loading rate, fibre-reinforced concrete absorbs more impact energy than normal concrete [21, 22]. Although the use of energy-absorption materials such as fibre and RA significantly enhanced the energy-absorption capacity of concrete-like materials, it also reduced workability. SCC is widely used for its excellent workability [18, 23]; thus, there is a need for an alternative material that can improve the energy absorption property of SCC without compromising its workability [23, 24].

Therefore, some researchers consider the employment of polymers to enhance the mechanical properties of SCC and describe that SCC with enhanced strength, low dynamic elastic modulus, and improved durability could be formed through the integration of polymers [25, 26]. SCC with improved performance properties can be achieved by incorporating polymers, but the cost increases accordingly. That is, polymers could excellently modify the properties of SCC, but they were costly. Therefore, a more economical material is needed for this purpose. Some previous studies have suggested that asphalt is a good material for the required properties of SCC [24, 27]. In addition, asphalt emulsion (AE) is less expensive, and because of its flexible nature, it can effectively enhance the toughness and durability of SCC [11]. Previous studies on the use of AE in cement-based materials have indicated that at the same loading rate, cement asphalt emulsion mortar (CA mortar) is more sensitive to strain rate than concrete [28, 29]. Zhang *et al* [30] reported that an appropriate dosage of AE enhanced the durability of SCC without adversely affecting its workability. Li *et al* [27] investigated the dynamic mechanical properties of SCC designed to incorporate asphalt-coated coarse aggregate (ACCA) and found that the incorporation of ACCA greatly enhanced the impact resistance and impact toughness index of SCC. However, the impact toughness index largely depends on the thickness of the asphalt layer on the aggregate surface. Additionally, previous studies

indicated that recycled asphalt pavement (RAP) can be employed to produce SCC with appropriate workability and improved permeability properties [31–33]. Although SCC with appropriate properties has been produced by incorporating RAP and ACCA, the literature on the use of AE in SCC and its effects on both fresh and hardened properties remains very limited. Because AE is liquid at room temperature, it can be easily incorporated into concrete; thus, integrating AE into SCC might be cost-effective and less challenging than using RAP and ACCA.

In the pursuit of optimizing complex systems and processes, scholars and engineers often encounter numerous variables that influence the desired outcome. Response surface methodology (RSM) has been a powerful and systematic approach to tackle this challenge, enabling researchers to explore the intricate relationships between input variables and the response of interest [34, 35]. RSM is a collection of statistical and mathematical techniques used to model and analyse the relationship between a set of independent variables and the output response of a system. It is commonly used in scientific research, engineering, and manufacturing to improve and optimize the performance of complex systems [34, 36, 37]. It operates on the premise that a suitable mathematical model can be developed to represent the underlying relationships by systematically varying the input variables and observing the corresponding responses. This model, often expressed as a response surface, can be employed to optimize the system, predict outcomes, and guide decision-making [35, 38]. The versatility of RSM enables researchers to optimize processes, design experiments, and gain deeper insights into complex systems [39, 40].

RSM stands out as an incredibly effective approach due to its economical use of partial factorial designs, such as star designs or CCD. Consequently, minimal experimental data points are required [41, 42]. Among the prevalent first-order designs in RSM are the Plackett–Burman, simplex designs, and 2k factorial, while the Optimal (custom) design, Definitive Screen (DSD), Box–Behnken, and Central Composite are the typical examples of widely used second-order designs [39, 43]. For this study, AE was integrated into the SCC; the workability, quasistatic compressive strength, and pore structure were studied. The compressive stress–strain curves were discussed, and the toughness index of SCC with AE was evaluated. The pore structure of the SCC with AE was studied using microscopy image analysis. Scanning electron microscopy (SEM) and X-ray diffraction (XRD) tests were also performed to discuss the effects of AE on the microstructural development of SCC. This research work provides both theoretical and technical guidance on using AE to improve the performance of SCC.

2 Materials and methods

2.1 Materials

Ordinary Portland cement (OPC), P.O 42.5 (C), which complies with the Chinese specification GB175-2007 [44], was adopted as the binder. Other cementitious materials, such as fly ash (FA), ground granulated blast furnace slag (GGBS, named SL), viscosity-modifying agent (VMA), and expansion agent (EA), were blended into the binder to improve the rheological and mechanical performance of the self-compacting concrete (SCC). Thus, the cementitious binder materials used in this study comprise cement (C), FA, SL, VMA and EA. The physical characteristics and chemical composition (oxide) of the cementitious materials, as

measured by X-ray fluorescence (XRF), are shown in Table 1. Crushed gravel (G) with a maximum nominal size of 5–20 mm and apparent density of 2680 kg/m³ was adopted as the coarse aggregate, whereas the fine aggregate was natural river sand (S) with an apparent density of 2650 kg/m³, fineness modulus of 2.6, and maximum particle size of 4.75 mm. The grading and quality specifications for the aggregates for the production of SCC were also met. A cationic asphalt emulsion (AE) with 60% residual asphalt was utilized as the modifier for improving the toughness and energy absorption capability of SCC. The main physical characteristics of the AE are presented in Table 2. A polycarboxylate-based superplasticizer (SP) with a water-reducing ability of approximately 26% was utilized to obtain the desired workability while guaranteeing the mixture stability. The mixing water (W) of all concrete mixtures was potable tap water. To decrease the entrapment of air induced by the mixing of asphalt emulsion and guarantee a homogeneous microstructure in the ITZ and concrete mixture, silicone-type defoamer (D) was also added to the mixtures at a dosage of 0.1% with respect to the total mass of cementitious materials. The choice of these materials ensured the feasibility of the fabrication of SCC with good fresh status stability and provided an adequate framework

for thoroughly analysing the effect of AE on the mechanical behaviors, toughness, and microstructure of hardened concrete.

2.2 Mix proportion

Five different self-compacting concrete (SCC) mixes were formulated to investigate, in a systematic way, the effect of asphalt emulsion (AE) addition on the fresh, mechanical, and microstructural characteristics of SCC. An experimental plan consisted of one mix without AE as a reference (SC0) and four mixes containing 5%, 10%, 15%, and 20% AE (SC5, SC10, SC15, and SC20), respectively. AE was used as a percentage of the total amount of cementitious materials. Other mix materials, proportions of constituents (aggregate grading and binder composition), as well as the water/binder (w/b) ratio, were kept the same for all the mixes to avoid other factors influencing the mixes' performance instead of AE. The w/b ratio of 0.34 was selected because it allows suitable workability conditions as well as an equivalent level of hydration of SCC mixes, thus being able to cover SCC requirements for workability. The mix proportion details of the five SCC mixtures are given in Table 3.

Table 1. Physical and chemical properties of cementitious powder materials

Oxides	Materials			
	C	FA	SL	EA
SiO ₂	19.86	55.70	26.10	6.00
Al ₂ O ₃	4.81	28.39	13.80	3.26
Fe ₂ O ₃	4.67	4.69	14.10	0.94
CaO	59.48	3.42	33.60	60.85
MgO	2.78	0.75	8.10	2.83
SO ₃	2.68	0.88	-	24.21
eq-Na ₂ O	0.65	0.95	0.45	0.80
Loss on ignition	2.40	4.70	2.10	6.60
Specific surface area (m ² /kg)	332	450	420	320

Table 2. Properties of Cationic Asphalt Emulsion

Parameter	Unit	Value
Penetration (25 °C, 100 g)	0.1 mm	82
Storage stability (1d, 250 °C)	%	0.35
Polarity	-	Cationic
Ductility (15 °C)	cm	52
Solubility (trichloroethylene)	%	97.2
Residue on screen (1.18 mm)	%	0.02
Solid content	%	60

Table 3. SCC mix proportions with AE (kg/m³).

Mix ID	C	FA	SL	EA	VMA	W	SP	S	G	AE	D
SC0	302	62.5	62.5	42	31	170	10.5	835	834	0	0
SC5	302	62.5	62.5	42	31	160	10.5	835	834	25	0.5
SC10	302	62.5	62.5	42	31	150	10.5	835	834	50	0.5
SC15	302	62.5	62.5	42	31	140	10.5	835	834	75	0.5
SC20	302	62.5	62.5	42	31	130	10.5	835	834	100	0.5

C: cement; FA: fly ash; SL: slag; EA: expansion agent; VMA: viscosity-modifying admixture; W: water; SP: superplasticizer; S: sand; G: gravel; AE: asphalt emulsion; D: defoamer.

2.3 Sample preparation

The SCC mixtures were prepared using a 60 L pan concrete mixer in a standard mixing procedure to maintain an even distribution of the asphalt emulsion (AE) in the mixtures and to guarantee the uniformity of the whole batch. The mixing process started with blending the dry materials: cementitious constituents, coarse and fine aggregates, for 30 s at a speed of 50 rpm to reach a homogeneous dispersion of the materials. Next, the predetermined amount of mixing water was added gradually while mixing was maintained. Afterwards, the asphalt emulsion defoaming agent (AE-D) solution, premixed in an appropriate amount, was slowly added and mixed for approximately 2 min to obtain the total dispersion of AE in the cementitious mixture. Finally, the prescribed content of the polycarboxylate-based superplasticizer (SP) was added to reach a certain degree of self-compactability. The measured fresh properties of the SCC mixtures: slump flow and J-ring test performed just after mixing in accordance with the SCC tests are given in Table 4. The whole testing was conducted in controlled laboratory conditions at ambient temperature between 20–25°C. Once the fresh-state testing was complete, the SCC mixtures were poured into moulds of suitable dimensions to facilitate each mechanical and microstructural test needed. Entrapped air was expelled without disturbing the self-compacting ability of the concrete by gentle tapping of the moulds where required (without external vibration). The moulds were covered to minimize moisture loss, then demoulded after 24 h and placed in a standard curing chamber to be cured at 20±2°C and 95–100% relative humidity until the respective testing ages.

2.4 Experimental methods

2.4.1 Workability properties test

In the fresh state, slump flow and J-ring tests were performed to assess the flowability and passing ability of the

self-compacting concrete (SCC) mixes, respectively [45]. In the slump flow test, fresh SCC was introduced into a slump cone, and the overflow was levelled with the slump cone and pulled up. Then, two average mutually orthogonal slump spread values were registered (SF), along with the time taken to reach a spread of 500 mm (T) as a measure of viscosity (Fig. 1a). In the J-ring test, a J-ring was placed at the center of the slump cone while casting fresh SCC, simulating reinforcement bar congestion. Once the concrete had stopped spreading, the average of two mutually orthogonal slump spreads was measured (JF) (Fig. 1b). The difference between the SF and JF also indicates the passing ability of SCC, and the lower the value is, the higher the passing ability of concrete to overcome obstacles without getting stopped. This test programme will ensure adequate comparison of the fresh concrete and self-consolidating behaviour of the SCC mixture with variations in asphalt emulsion content.

2.4.2 Quasistatic compressive test

A compressive property test was carried out after 56 days of curing using a servo-hydraulic testing machine (Instron 1342) under quasistatic uniaxial compressive loading according to the Chinese Standard GB/T 50081-2002 [46]. Figure 2 shows the experimental setup and cylindrical samples (Φ75 mm × 75 mm). The test was performed in displacement control at a rate of 0.15 mm/min, approximately 3.0 10 s. To avoid interface friction, petroleum jelly was spread on specimen surfaces. An extensometer was used to record the longitudinal displacement. Three replicates were performed and averaged for each mix. Based on the stress–strain curve results, the compressive behavior and energy absorption ability of the AE-modified SCC mixture were assessed. Energy absorption capability, termed toughness, can be calculated as the area under the stress–strain curves [47–49]. As the indicator for direct quantitative analysis of total toughness, the toughness index (T) suggested by Khaloo *et al.* [50] was selected, as shown in Fig. 3. The

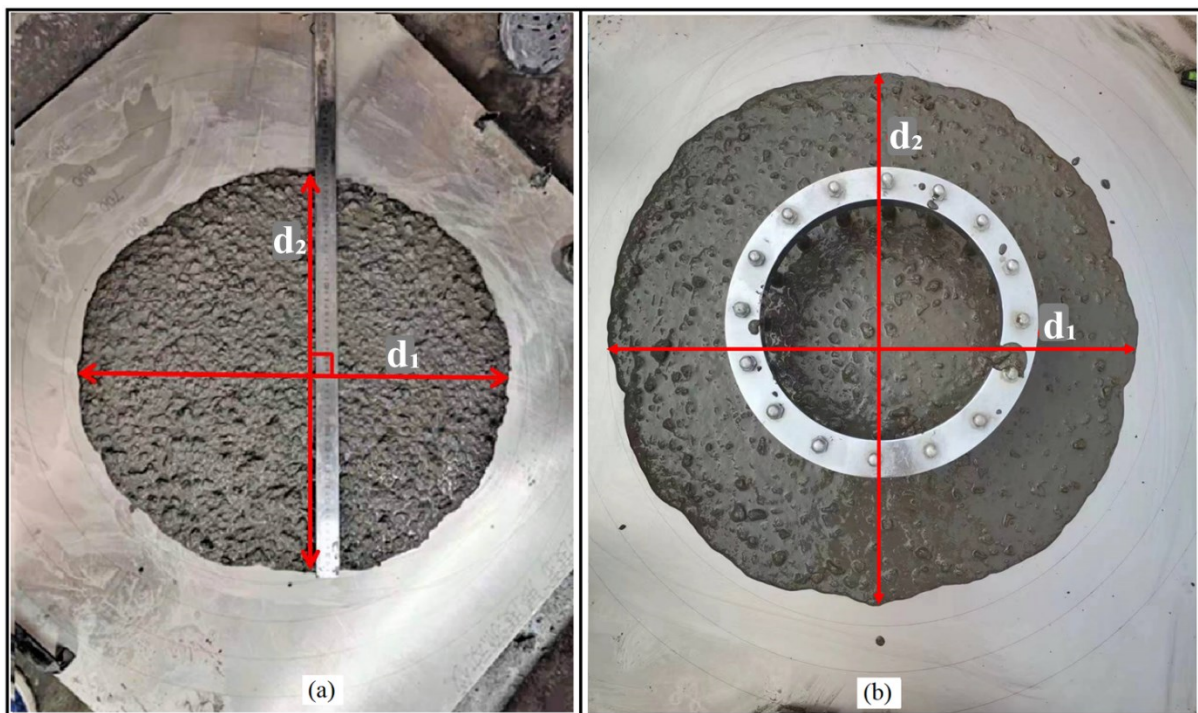


Figure 1. Workability of SCC: (a) slump flow; (b) J-ring flow

equation for calculating the toughness index T is shown in Equation (1). The toughness index T integrates the combined effects of compressive strength and deformability into one value.

$$T_i = \frac{T_{80}}{T_{100}} \quad (1)$$

where T_{80} represents the region under the stress–strain curve up to 80% of the maximum stress (illustrated by the sum of S_A and S_B in Fig. 3); and T_{100} is the region under the stress–strain curve up to the highest stress (illustrated by S_A in Fig. 3).

2.4.3 Pore structure test

The pore structure of the hardened concrete specimens was studied via the pore structure analyser CABR-457, designed and manufactured by the China Academy of

Building Research, as shown in Fig. 4. After 56 days of standard curing, a slice piece of concrete with dimensions of 100 mm × 100 mm × 20 mm was cut from the concrete cube specimen (100 mm × 100 mm × 100 mm) for this test. A clean, smooth surface was selected for the image observation, and it was polished again with a grinding machine to enhance its smoothness and clarity, ensuring optimal conditions for observation. After grinding, an ultrasonic cleaner was used to clean the polished test blocks, which were then oven-dried at approximately 55°C. After drying, the observation surface was highlighted with a marker, and the drying process continued for a few minutes. Subsequently, the specimens were removed from the oven, and the observation surface was coated with a heated mixture of zinc oxide and Vaseline, ensuring that the white paste (a mixture of zinc oxide and Vaseline) effectively filled all the pores on the observation surface. Excess paste was scraped off after cooling to room temperature. The test

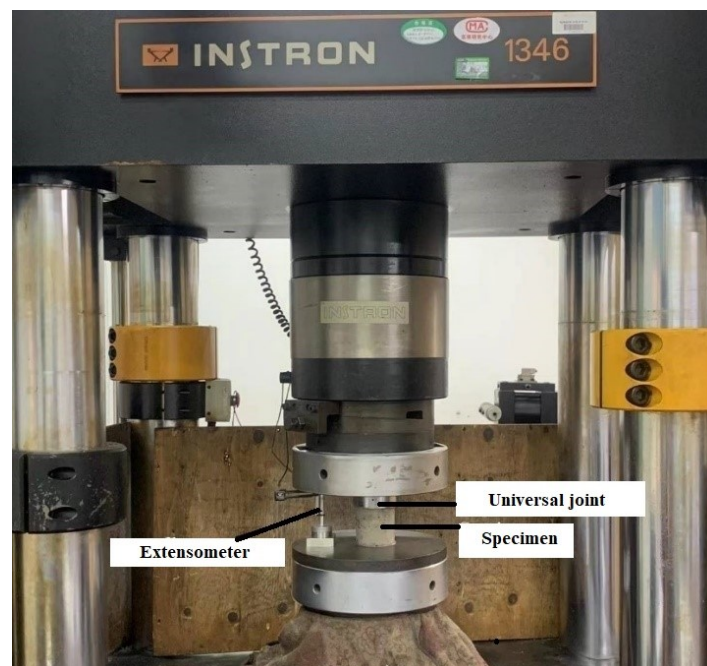


Figure 2. Quasistatic compression test

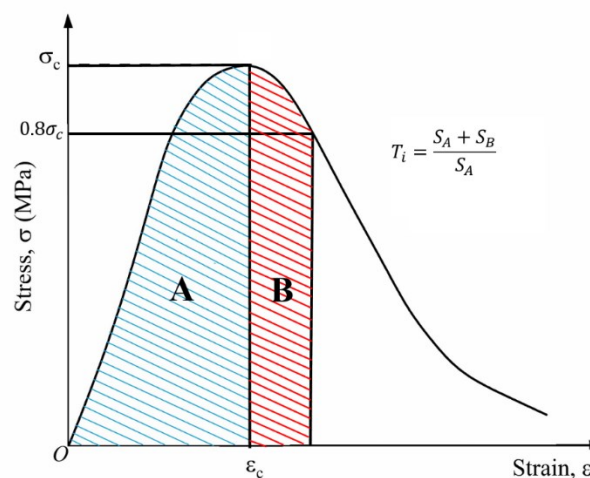


Figure 3. Definition of toughness index

specimen was then put on the stage and observed using a pore structure analyser, with the observation surface exposed to the microscopic camera. Before the observation started, the camera focus was adjusted to obtain a clear image of the test surface. During the observation, the white pixels in the image were identified by the computer as pores. Initiating from the upper left corner of the specimen, the observation surface was scanned five times along the X and Y axes, and from the images captured by the computer, the pore size distribution and porosity could be computed automatically using a C++ software design language based on the following equations [51]:

$$A_i = \sum_{j=1}^{m_i} A_j \quad (2)$$

$$D_j = \sqrt{\frac{4A_j}{\pi}} \quad (3)$$

$$A\% = \frac{\sum_{i=1}^{250} A_i}{250A_T} \quad (4)$$

where A_i = total pore area (cm^2) observed in the i th microscopy images A_j = area of the j th pore (cm^2) D_j = equivalent diameter of the j th pore (cm^2) A_T = total analysed area (cm^2) of each microscopy image. $A\%$ = total pore area ratio (%) of the sample. Three regions were subjectively chosen from each sample, and average results were considered to guarantee statistical stability.

2.5 Response surface methodology-based modelling

RSM software (i.e., Design-Expert 13) was used in this study to analyse experimental data using the optimal (custom) method, enabling statistical analysis and model development to predict the effects of AE and curing period on the properties of self-compacting concrete. For the purpose of model development, the compressive strength test was conducted at different curing periods; three cubes ($100 \text{ mm} \times 100 \text{ mm} \times 100 \text{ mm}$) were tested from each group, following Chinese standard GB/T 50081-2002 [46], the average of the three results was recorded. The experimental results from the compressive strength tested at different curing periods and that of workability test was then used as input parameters. The regression model for the optimal calculation of the input parameter in this study is given by Eq. (5).

$$y = \beta_0 + \sum_{i=1}^n \beta_i X_i + \sum_{i=1}^n \beta_{ii} X_i^2 + \sum_{i < j} \beta_{ij} X_i X_j + \epsilon \quad (5)$$

where y is the response, β_0 is the intercept, β_i and β_{ii} are the first- and second-order coefficients, β_{ij} is the interaction coefficient, X_i and X_j represent the independent variables, and n and ϵ represent the number of variables and the model's error, respectively [52].

3 Results and discussion

3.1 Flowability

Table 4 shows the fresh properties of SCC mixtures incorporated with different amounts of asphalt emulsion (AE). In general, as AE increased, the flowability and passing ability of SCC decreased, while the passing performance worsened with increased AE content. As seen in Table 4, slump flow values decreased as the amount of AE increased. This decrease in slump flow is due to the lower amount of free water, since part of the total water used in the mixtures was provided by asphalt emulsion, as shown in Table 3; free water for lubrication in the SCC mixes decreased. Additionally, AE can hinder cement hydration and the increase in fresh matrix viscosity that causes cement paste, as well as aggregate movement [30, 53]. With a reduction in flowability for SCC with 5 and 10% AE relative to the control mix, a greater flow loss occurred for specimens prepared with 15 and 20% AE. T_{50} flow values increased with increasing AE content, as shown in Table 4, from 6.7 s (SC0) to 10.5 s (SC20), which is in line with the observed trend of decreased flowability. The maximum T_{50} value was obtained in SC20, which meant that the AE content significantly increased the viscosity of SCC and delayed its flow due to self-weight. Additionally, the difference between the SF and JF values increases with the AE dosage, which proves that the mixture has a gradual decrease in the passing ability when the dosage is increased. This trend becomes remarkable in the 15% and 20% AE mixtures and indicates high flow restrictions within a reinforcement model and a strong possibility of blocking the concrete. However, the results are in accordance with the requirements of passing ability in ASTM C1621 [54], and the introduction of AE to the fresh concrete mix with AE dosage up to 10% does not compromise the self-compacting behaviour of concrete.

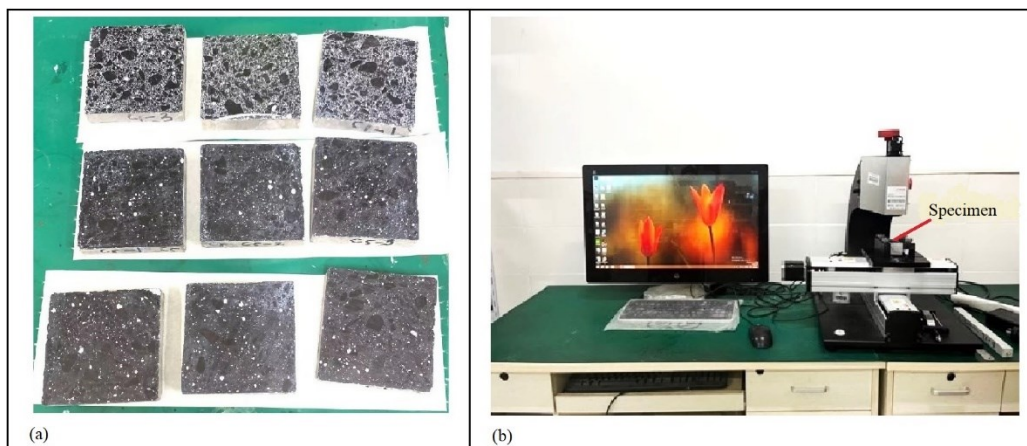


Figure 4. Pore structure test: (a) prepared specimens; (b) pore structure analyser

From the above analysis, we found that asphalt emulsion can enhance the fresh concrete viscosity and moderately affect the workability of the concrete mixtures. Workability in mixes with 10% AE is still acceptable, with the benefits of mixture stability compared to higher AE percentages. The addition of more AE content would affect the fresh performance of the mixture badly, but not in an equitable return.

3.2 Quasistatic compression test

The 56-day quasistatic compressive stress–strain curves of SCC containing different amounts of AE are presented in Fig. 5(a). From these curves, it can be seen that as the AE content increases, the compressive strength declines, while the deformation capacity of the concrete gradually improves. The peak stress declined as the AE content rose, whereas the peak strain increased, suggesting a shift towards a more ductile behaviour rather than a brittle behaviour. When the AE content reached 5%, 10%, 15%, and 20%, the compressive strength decreased by 7.87%, 9.21%, 28.10%, and 40.97% compared to the control mix, respectively. The strength loss of SCC was most evident as the AE content increased above 10%, implying that too much asphalt emulsion would detrimentally influence the load-carrying capacity of SCC. In this investigation, strength reduction is attributed mainly to the influence of the asphalt emulsion on the cement hydration process. Asphalt emulsion, in the presence of cement, began to demulsify and form a film of cohesive asphalt that encapsulated unhydrated cement particles, cement hydration products, and the surface of aggregate. The asphalt films partially inhibit cement hydration and form a thin film around the ITZ to impede further formation of hydrate and result in weakening the bonding at the ITZ between aggregate and cement mortar. Therefore, the compressive strength of the composite decreased. This conclusion is in agreement with other research [55, 56], stating that either the internal cohesion of asphalt or weakened asphalt–cement bonds play a decisive role in weakening asphalt-modified cementitious composites. Although the peak stress decreased, the postpeak response of the AE-modified mixtures showed a distinctly smoother postpeak slope in the descending branch of the stress–strain curve compared with the control SCC. This gentler postpeak slope reveals higher plastic deformation and slow crack propagation, which represent a ductile mode of failure. Therefore, asphalt emulsions have been demonstrated to improve the deformability and energy dissipation capacity in the postpeak region for SCC.

However, these improvements come at the cost of compressive strength, which is most apparent in the higher concentrations of AE. The study indicated that AE added at a concentration of 10% has the best balance between strength and deformability, which could be useful in some projects required to have a better resistance to stress and crack damage.

3.3 Toughness index

Fig. 5(b) shows that AE also affects the toughness index of the SCC mixtures. As can be observed, the toughness index has a tendency to increase as the dosage of AE increases, which shows that AE is indeed effective for improving the energy absorption capacity of SCC. The toughness indices are 5.21%, 17.00%, 24.13%, and 39.52% higher than that of the control mixture for specimens containing 5%, 10%, 15%, and 20% AE, respectively. This increase in toughness is mainly because asphalt has a viscoelastic property, which allows the improved cementitious matrix to undergo a large deformation while absorbing a large portion of mechanical energy when compression is applied.

During the process of demulsification of the asphalt emulsion, connected asphalt films will form in the matrix to bond the interface of hydration products and aggregates, which can function as a more elastic phase to postpone the cracking initiation, propagation, and distribution and postpone the further degradation capacity of the SCC after peak [27, 57]. Another benefit of having an asphalt film on the surface of the aggregates is to minimize stress concentration on the interface of aggregates and pastes and diminish the wedging effect of aggregates under loading. Thus, the stresses are better distributed in the matrix, which helps improve the deformation energy absorption of materials before failure. Similar toughening mechanisms are well documented for asphalt-modified cementitious materials in other related research [27, 57]. Even though the increase in toughness is most pronounced for AE content close to 20%, a sharp decrease in compressive strength resulted for high AE content. Regarding the strength, toughness, and practical use for structural applications, the optimal AE content seems to be approximately 10%. With such an amount of AE, the increase in toughness reaches as high as 17.00%, while the compressive strength and self-compactability are reasonable and applicable for engineering purposes.

Table 4. Results for fresh properties

Mix ID	Slump flow (mm)	J-ring flow (mm)	Δ SF (mm)	T_{50} (s)	Bulk density (kg/m ³)
SC0	630	622.5	7.5	6.7	2176
SC5	617	609.1	7.9	7.8	2301
SC10	620	612.2	7.8	7.9	2302
SC15	585	570	15	8.7	2296
SC20	562	543	19	10.5	2273

Note: T_{50} = slump flow rate; Δ SF = difference between slump flow and J-ring flow

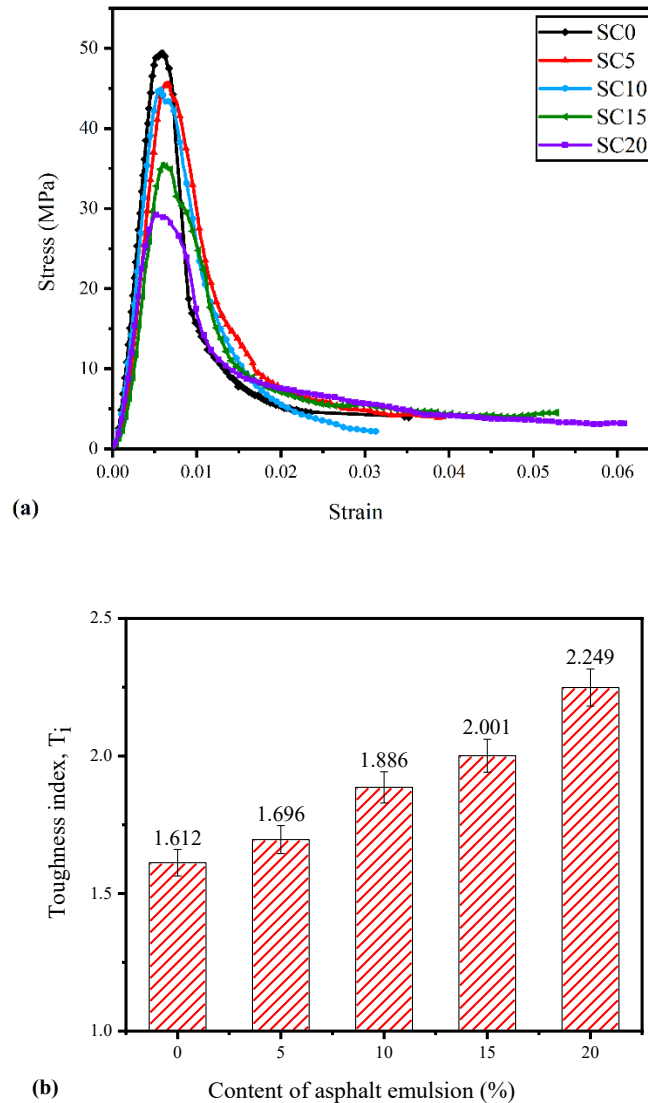


Figure 5. (a) Quasistatic stress–strain curves; (b) toughness index.

3.4 Pore structure

Fig. 6 presents the pore structure of the SCC specimens. The average pore diameter of the SCC decreases as the AE content increases. Specifically, the average pore diameter of the specimen with 5% AE is approximately 76% of that of the control specimen. When the AE content was increased to 10%, 15%, and 20%, the average pore diameter decreased further to 67%, 62%, and 57%, respectively, compared to the control set. This effect is also reflected in an increase in the pore spacing factor, as illustrated in Fig. 6(a), which is the maximum distance from any point within the hardened specimen to the nearest air void. The pore size distribution presented in Fig. 6(b) reveals that the pore content in the size range from 0 to 100 μm increased with the addition of AE, while the pore content in the size range from 100 μm to 1000 μm decreased with the addition of AE. Additionally, the content of pores with sizes ranging from 1000 μm to 3000 μm increased slightly as the AE content increased. That is,

the number of tiny pores increases, the number of medium pores decreases, and the number of large pores increases. The addition of AE retards cement hydration, resulting in fewer hydration products and a looser, more porous microstructure. However, free asphalt particles flow into the pores of the matrix, especially the medium-sized pores, which is why the pore content decreases in the range of 100 μm to 1000 μm . The increase in large pores due to AE contributes to improved toughness and decreased compressive strength, while the reduction in medium pores due to the filling effect of AE accounts for the decreased permeability of the SCC matrix. Moreover, it is important to note that the pore distributions for the control specimen are mainly within the range of 100 μm to 1000 μm , whereas a large percentage of pores in the AE samples are distributed within the range of 0 to 400 μm . As a result, the average pore diameter decreased, and the pore spacing factor increased for the groups with AE.

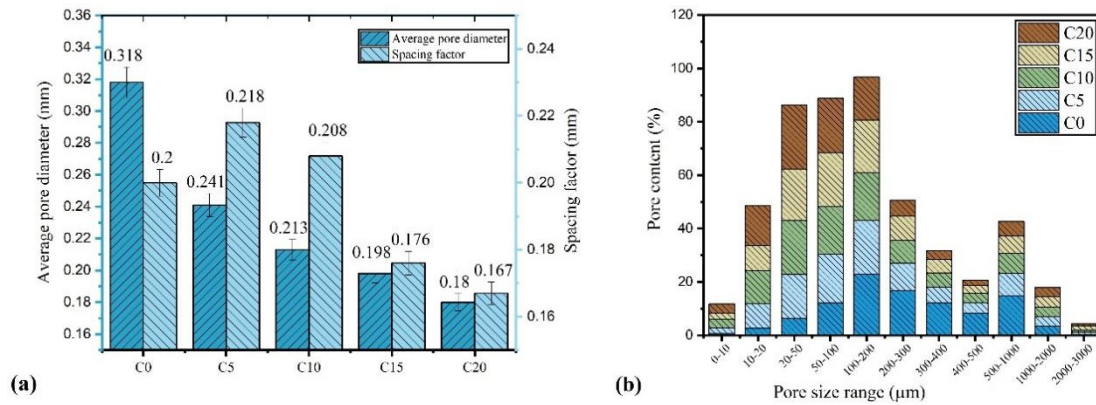
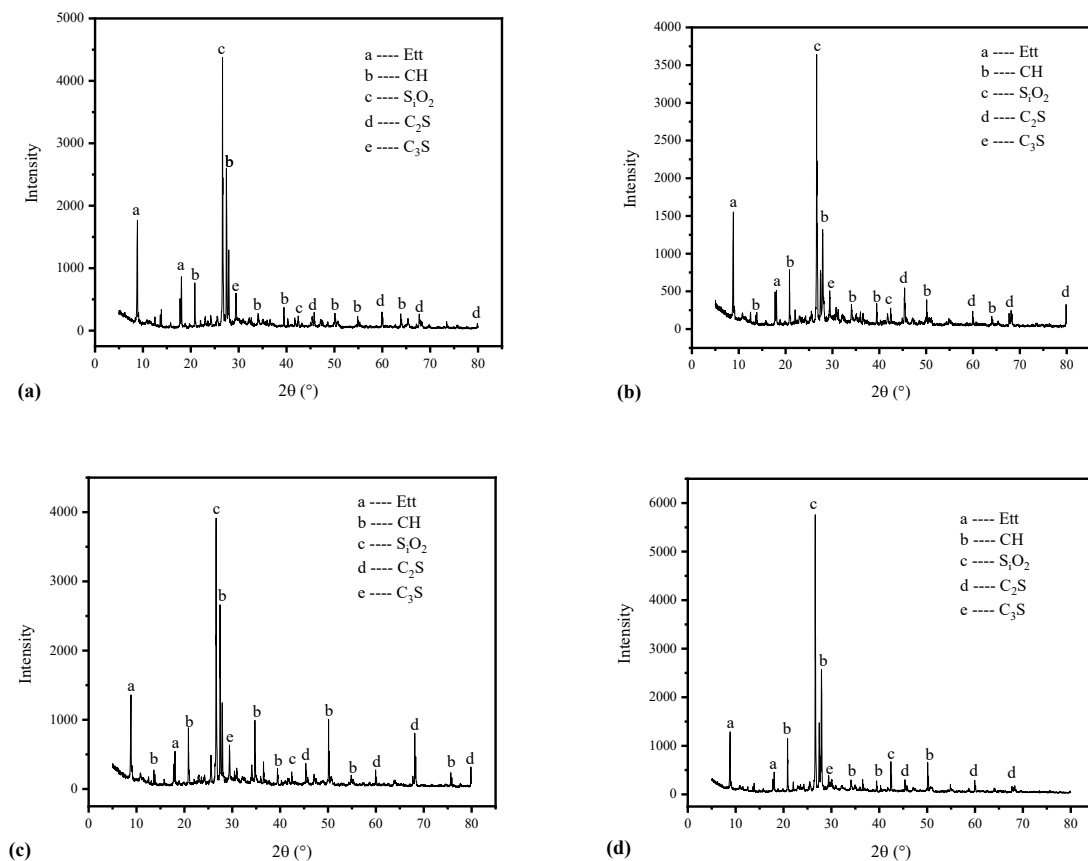


Figure 6. Pore structure of SCC specimens: (a) average pore diameter and spacing factor, (b) pore size distribution

3.5 X-ray diffraction (XRD) analysis

X-ray diffraction (XRD) analysis of the SCC specimens was carried out to determine the phase composition and qualitatively analyse the effect of AE on the hydration products of SCC. Fig. 7 presents the XRD results for SCC specimens with varying AE contents. The main products of cement hydration identified include ettringite (Ett), portlandite (CH), SiO_2 , C_2S , and C_3S . The figure shows that the XRD spectra of the specimens exhibit similar patterns as AE increases, with small discrepancies in trend. On average, the peaks of CH, C_2S , and C_3S decreased with the addition of AE. For example, in the spectra of C0, C5, and C10, the peaks were easily identified, whereas in the spectra of the specimens with 15% and 20% AE (SC15 and SC20), some

peaks disappeared. This can be attributed to the retarding effect of AE on cement hydration [58]. It was observed that with the addition of AE content, the hydration degree of cement decreases, thus resulting in a reduction in the amount of hydration products. However, according to the XRD results, it seems that the hydration products of the AE-modified specimens are almost the same as those of the reference mix. This result indicated that AE incorporation could not change the mineralogy and the type of hydration products [30] but could mainly reduce the hydration degree of cement, thus decreasing the amount of cement hydrate while retaining the original hydration mechanisms, and such a phenomenon accords with the results obtained from the compression test.



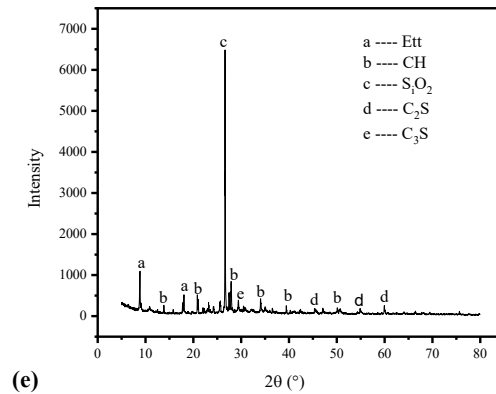


Figure 7. XRD of SCC samples with different amounts of AE (a) SC0 (0% AE); (b) SC5 (5% AE); (c) SC10 (10% AE); (d) SC15 (15% AE); (e) SC20 (20% AE).

3.6 Compressive strength

The compressive strength of the specimens at different curing periods is presented in Fig. 8. The figure indicates that asphalt emulsion deteriorates compressive strength of SCC specimens. However, the rate of the strength reduction declines with curing time; thus, the compressive strength improves with curing period. After 14 days of curing, the compressive strength decreased within the range of 15.6% – 39% due to the addition of 5 – 20% AE, respectively. After 28-day curing, the reduction in strength was within the range of 11.5% – 30.7%. Despite the effect of AE, adequate strength is achieved for all groups after 56 days of curing. The strength reduction is attributed to the reterding effect of AE on hydration process, which inhibits further hydration and, consequently, affects the formation of hydration products. Thus, with the addition of asphalt emulsion, the amount of hydrates produced during hydration of cement

decreased, which eventually led to the decrease in compressive strength [59].

3.7 Mathematical modelling

The accuracy of the model and the significance of the variables and their interaction with the response variables are usually evaluated using analysis of variance (ANOVA). After inputting experimental data and performing regression analysis, the RSM identified the quadratic models as the most appropriate for predicting the flowability and compressive strength of SCC. The proposed model equations for all the responses are given in Eqs. (6) and (7). The responses were determined using the proposed models. Table 5 presents ANOVA results for the effects of AE and curing time on flowability and compressive strength; Table 6 presents the models' validation results.

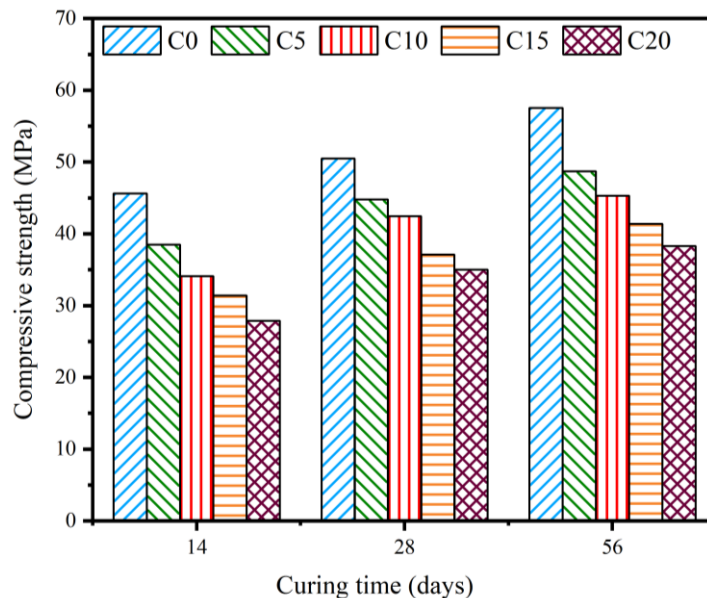


Figure 8. Compressive strength

$$SF = 627.985 + 0.694x - 0.2x^2 \quad (6)$$

$$CS = 35.818 - 1.321x + 0.729y + 0.000159xy + 0.0229x^2 - 0.00705y^2 \quad (7)$$

where SF and CS are the responses representing slump flow and compressive strength, respectively; x represents the content of the asphalt emulsion; and y is the curing time.

3.7.1 Verification of the proposed models

The models were verified using ANOVA at a 95% confidence level (i.e., 5% significance). Hence, any model or model term with a p value < 0.05 is statistically significant. The ANOVA results in Table 5 indicate that all proposed models are significant, with p values < 0.05 . For the model terms, x and x^2 are all significant for slump flow; for compressive strength, x , x^2 , y , and y^2 are significant terms, while xy is not significant; this indicates that AE content and curing time independently affect the compressive strength of SCC, according to the developed model.

Furthermore, Table 6 presents the validation parameters for all responses; the p values are all less than 0.05, indicating statistical significance. The accuracy of the proposed models can also be assessed through the coefficient of determination (R^2). High R^2 values of 0.96 and 0.992 are observed for slump flow and compressive strength, respectively. This indicates a higher degree of correlation between the predicted and actual experimental results and the model's validity in predicting the responses with high accuracy [60, 61]. Additionally, the predicted coefficient of determination (Pred. R^2) values match well with their corresponding adjusted coefficient of determination (Adj. R^2).

In addition, the adequate precision value assesses the signal-to-noise ratio of a well-developed model, with a minimum threshold of 4 required to ensure robustness [35, 62]. In this study, it is observed that all the proposed models achieved an adequate precision value of more than 4. This suggests that the models possess a high level of robustness and can effectively be used to navigate the design space.

3.7.2 RSM plot analysis for the influence of variables

In RSM, surface plots offer significant advantages for assessing potential relationships among variables. Fig. 8 shows the 3D surface plots and predicted-versus-actual plots for the influence of AE and curing time on compressive strength. The 3D surface plot shown in Fig. 8(a) provides insights into the effects of input variables on the output responses. It uses contour lines to display variations in the response under different levels and combinations of the independent variables. A color gradient is used to show the magnitude of responses in 3D plots, with red indicating the highest value and blue the lowest [62, 63]. Interestingly, the patterns observed from the plots are consistent with the experimental results. For example, the 3D surface plot of compressive strength shows a sloping-flat profile, indicating a well-established interaction between input variables [35, 61]. This underscores that the compressive strength decreases linearly with increasing AE content and increases

Table 5. ANOVA results for the response parameters

Response	Source	SS	DOF	MS	F value	p value	Significance
Slump flow	Model	8637.83	2	4318.92	118.5	<0.00001	Yes
	x -AE	7375.54	1	7375.54	202.36	<0.0001	Yes
	x^2	1075.2	1	1075.2	29.5	0.0003	Yes
	Residual	364.48	10	36.45	-	-	-
	Lack of Fit	364.48	2	182.24	-	-	-
	Pure error	0.0	8	0.0	-	-	-
	Cor Total	9002.31	12	-	-	-	-
Compressive strength	Model	1012.82	5	202.56	250.1	<0.0001	Yes
	x -AE	686.61	1	686.61	847.72	<0.0001	Yes
	y -time	254.44	1	254.44	314.14	<0.0001	Yes
	xy	2.34	1	2.34	2.89	0.1202	No
	x^2	19.39	1	19.39	23.93	0.0006	Yes
	y^2	24.77	1	24.77	30.58	0.0003	Yes
	Residual	8.10	10	0.81	-	-	-
	Lack of Fit	8.10	5	1.62	-	-	-
	Pure error	0.0	5	0.0	-	-	-
	Cor Total	1020.92	15	-	-	-	-

SS = Sum of squares; DOF = Degree of freedom; MS = Mean square

Table 6. Response model verification

Response	Slump flow	Compressive strength
R^2	0.960	0.992
Adjusted R^2	0.951	0.988
Predicted R^2	0.943	0.962
Adeq. precision	22.8	52.0
S.D	6.04	0.9
Mean	605.77	40.88
C.V%	0.997	2.2

S.D. = Standard deviation; C.V. = coefficient of variance

concurrently with extended curing time. The 3D surface plot of slump flow could not be obtained because only one variable, AE content, was included. Therefore, to graphically verify the model's adequacy, the predicted-versus-actual plot and the normal probability plot of the residuals (normal plot) were analysed and are presented in Fig. 9 (a and b). As shown in the figure, the data points were closely aligned with a straight line, confirming the accuracy of the developed model and the normal distribution of the data. The predicted vs actual plot for compressive strength is presented in Fig. 8(b). These plots illustrate the distribution of data points along the line of fit, and the degree of linearity among these points defines the accuracy of the models. The symmetrical distribution of data points along the line of fit suggests a strong correlation between the response model results and the experimental results [35, 61].

4 Conclusions

An experimental study was systematically performed on the effect of AE on the fresh properties, mechanical properties, and microstructural properties of SCC. A comparative result proved that AE is a good alternative to improve the toughness and energy absorption of the SCC and can also maintain a good fresh property in moderate AE content. The following conclusions were made based on the results of the experimental investigation and microstructure study.

1. Incorporating AE in SCC reduces workability, although the addition of up to 10% AE does not show a significant detrimental effect on flowability and passing ability. A higher AE content markedly reduces the workability of SCC.

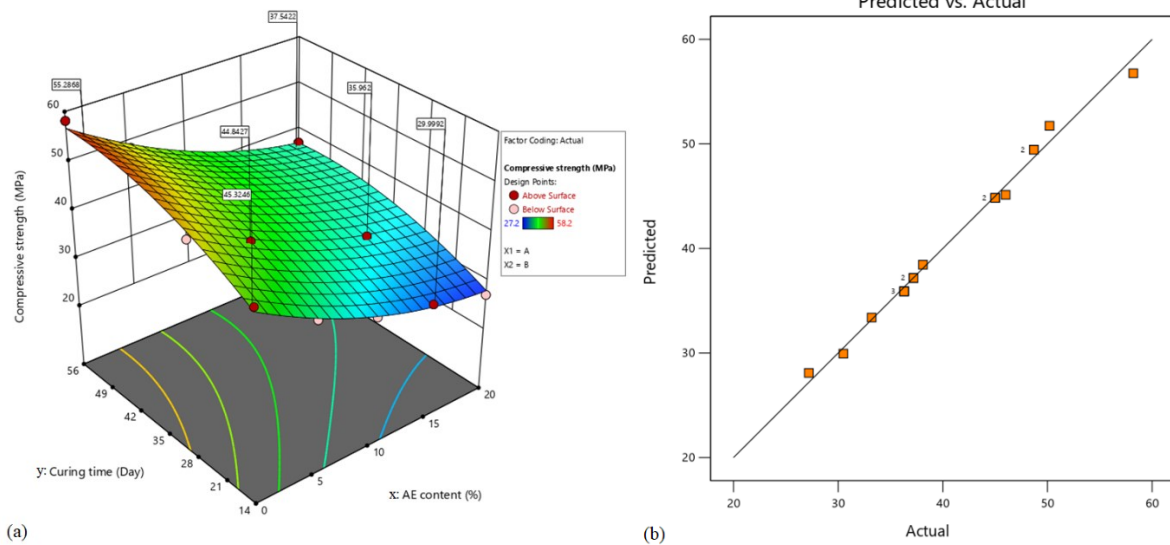


Figure 8. (a) Influence of AE and curing time on compressive strength: (a) 3D surface plot; (b) actual vs. predicted plot

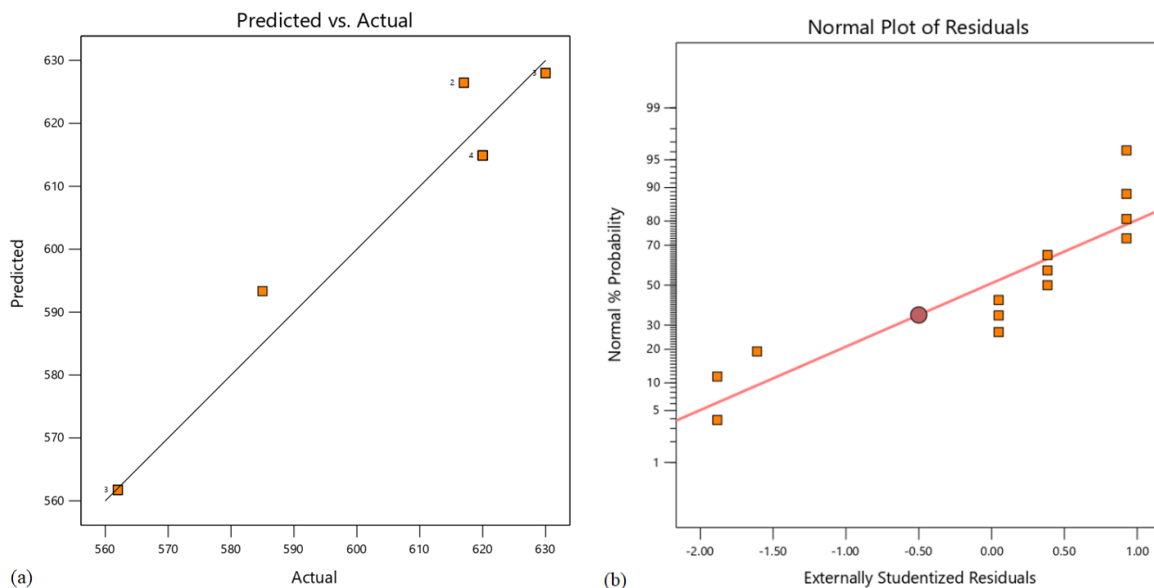


Figure 9. Effect of AE on slump flow: (a) predicted vs. actual plot; (b) normal plot of residuals

2. The addition of asphalt emulsion can decrease the peak compressive stress of SCC but also lead to a more ductile behaviour in terms of the development of the stress-strain curve: the postpeak part was much milder and more gradual, implying a much better deformation and energy absorption performance after postpeak fracture. This shows that the introduction of AE enhances the toughness and energy dissipation capacity of SCC, although the compressive strength is lower.

3. The toughness index of SCC specimens improves with the addition of AE. This implies that incorporating AE can effectively improve the impact resistance and energy absorption capacity of SCC.

4. The overflow asphalt particles penetrate and block up the pore system, which results in pore system refinement and improved compactness of the cementitious matrix. XRD tests indicate that the cement mineralogy and types of cement hydration products are not changed by introducing the asphalt emulsion; it decreases the hydration degree and thus the hydration product content but has no effect on the hydration mechanism.

5. Based on the experimental results from this study, the incorporation of 10% AE in SCC is recommended. This AE content has a favourable influence on energy absorption and toughness while maintaining the strength and excellent workability of SCC.

6. Mathematical models were established using RSM to predict the properties of SCC based on AE content and curing time. The models were validated using ANOVA, and all models achieved high R^2 (>0.9), a lower p value (<0.05), and strong agreement between the predicted and actual experimental results. Additionally, good agreement between the adjusted R^2 and predicted R^2 was observed, and the models had adequate precision greater than 4, indicating that they can be used to navigate the design space.

CRediT authorship contribution statement

Hussaini Abdullahi Umar and Ashiru Mohammed: Conceptualization; Data curation; Formal analysis; Investigation; Methodology; Roles/Writing - original draft. Luvuno Nkosingithi Jele, Aliyu Umar Mani, Ibrahim Aliyu, Zesizwe Ngubane, Jacob Adedayo Adedeji, and Nura Shehu Aliyu Yaro: Investigation; Methodology; Software; Validation; Visualization; and Writing - review & editing.

Declaration of conflict of interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgement

The authors acknowledge the support of the Harnessing Talent (HANT) Postdoctoral Fellowship (PDF) and Sustainable Environment and Transportation Research Group (SET-RG), Department of Civil Engineering, Midlands, Durban University of Technology, Durban 3021, South Africa.

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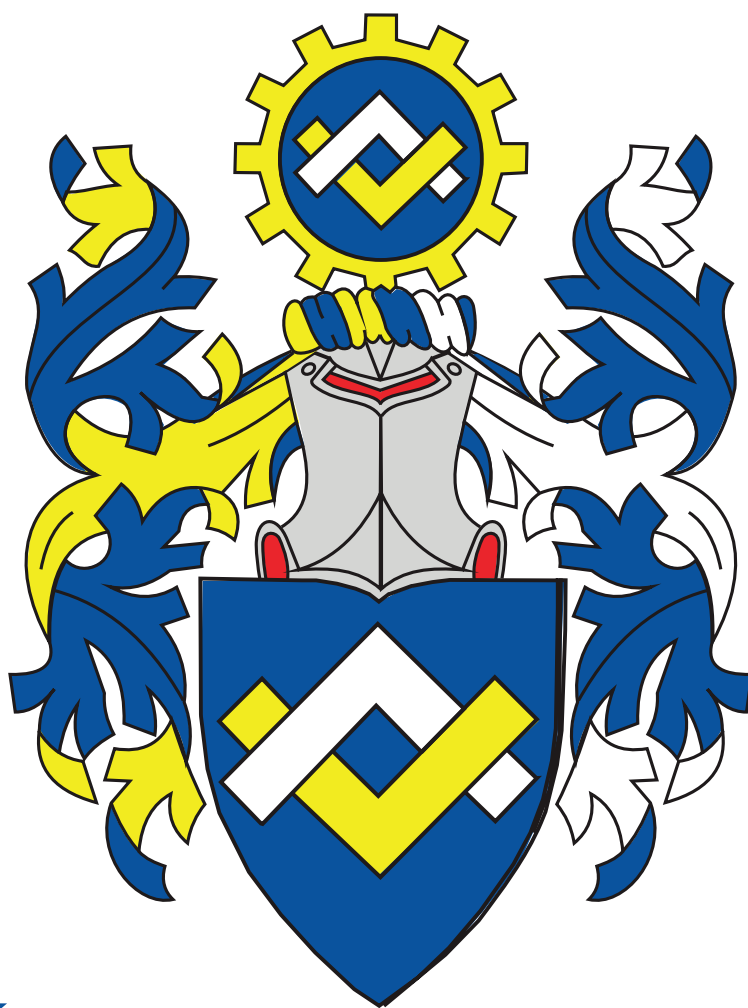


**MINISTRY OF EDUCATION, SCIENCE AND
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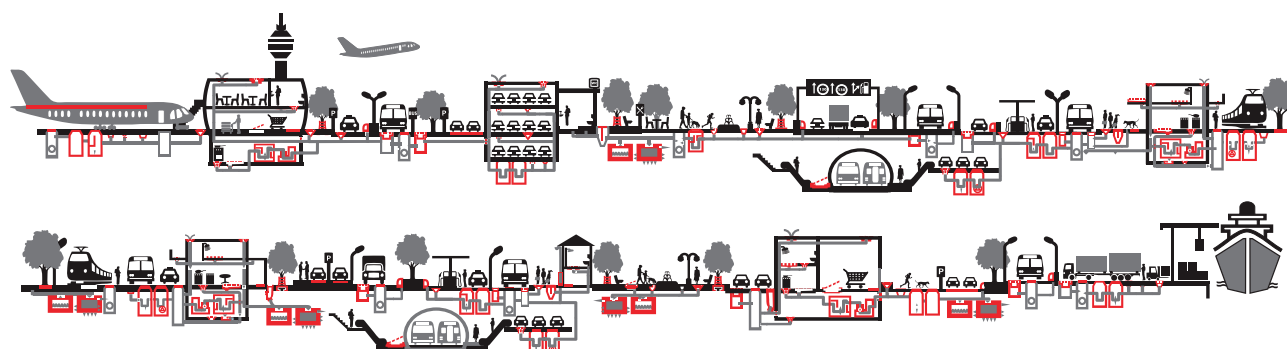
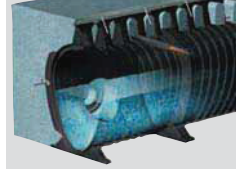
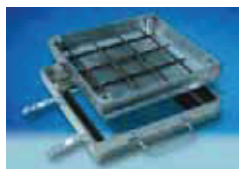
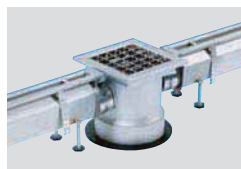
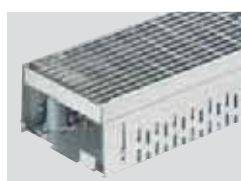
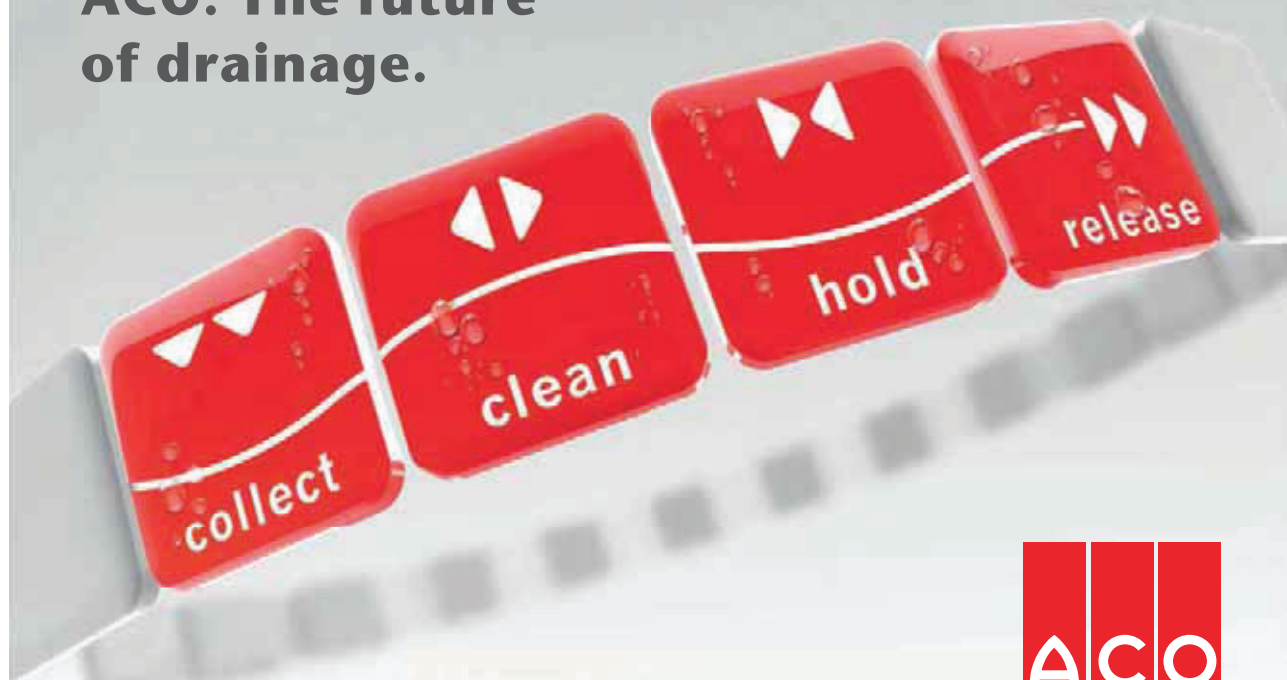
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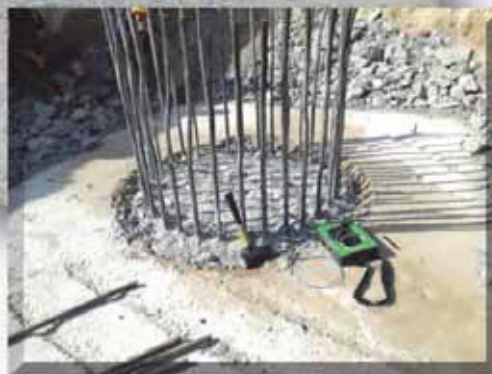
CENTAR ZA PUTEVE I GEOTEHNIKU

U okviru centra posluju odeljenja za geotehniku, nadzor i terenska ispitivanja, projektovanje saobraćajnica, laboratorija za puteve i geotehniku. Značajna aktivnost centra usmerena je ka terenskim i laboratorijskim geološko - geotehničkim istraživanjima i ispitivanjima terena za potrebe izrade projektno - tehničke dokumentacije, za različite faze i nivoe projektovanja objekata visokogradnje, niskogradnje, saobraćaja i hidrogradnje, kao i za potrebe prostornog planiranja i zaštite životne sredine. Stručni nadzor, kontrola kvaliteta tokom građenja, rekonstrukcije i sanacije objekata različite namene, izrada studija, ekspertiza, konsultantske usluge, kompletan konsalting u oblasti geotehničkog inženjeringa, neke su od delatnosti centra.



Ispitivanje šipova

- **SLT metoda (Static load test)**
- **DLT metoda (Dynamic load test)**
- **PDA metoda (Pile driving analysis)**
- **PIT (SIT) metoda (Pile (Sonic) integrity testing)**
- **CSL - Crosshole Sonic Logging**





- Ispitivanje šipova
 - Geotehnička istraživanja i ispitivanja – in situ
 - Laboratorija za puteve i geotehniku
 - Projektovanje puteva i sanacija klizišta
 - Nadzor



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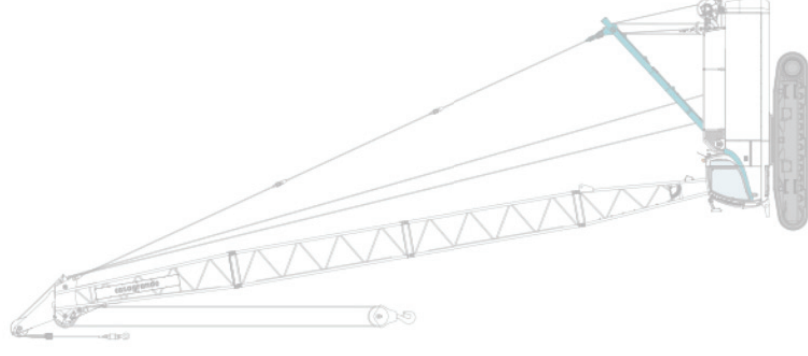


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PUT INŽENJERING



Put inženjering d.o.o punih 25 godina radi kao specijalizovano preduzeće za izgradnju infrastrukture u niskogradnji i visokogradnji, kao i proizvodnjom kamenog agregata i betona. Preduzeće se bavi i transportom, uslugama građevinske mehanizacije i specijalne opreme.

Koristeći inovativne tehnike i kvalitetan građevinski materijal iz sopstvenih resursa, spremni smo da odgovorimo na mnoge zahteve naših klijenata iz oblasti niskogradnje.



Osnovna prednost prefabrike konstrukcije jeste brzina kojom konstrukcija može biti projektovana, proizvedena, transportovana i namontirana.



Izvodimo hidrograđevinske radove u izgradnji kanalizacionih mreža za odvođenje atmosferskih, otpadnih i upotrebljenih voda, izvođenjem hidrograđevinskih radova u okviru regulacije rečnih tokova, kao i izvođenjem hidrotehničkih objekata.



Površinski kop udaljen je 35 km od Niša. Savremene drobilice, postrojenje za separaciju i sejalice efikasno usitnjavaju i razdvajaju kamene agregate po veličinama. Tehnički kapacitet trenutne primarne drobilice je 300 t/h.



Za spravljanje betona koristimo drobljeni krečnjački agregat sa našeg kamenoloma, deklariranih frakcija, kontrolisane vlažnosti. Kompletan proces proizvodnje i kontrole kvaliteta vršimo prema važećim standardima.



Obradu armature vršimo brzo, stručno i kvalitetno, sa kompjuterskom preciznošću i dimenzijama po projektu.



Naša kompanija u oblasti visokogradnje primenjuje sistem prefabrikovanih betonskih elemenata koji u odnosu na klasičnu gradnju ima brojne prednosti.



Prednapregnute šuplje ploče su konstruktivni elementi visokog kvaliteta, proizvedeni u fabrički kontrolisanim uslovima.



Izrađujemo betonske "New Jersey profile" koji se u svetu koriste za preusmeravanje saobraćaja i zaštitu pešaka u toku izgradnje puta, kao i Betonblock sistem betonskih blokova.



Uslugu transporta vršimo automikserima, kapaciteta bubnja od 7 m³ do 10 m³ betonske mase. Za ugradnju betona posedujemo auto-pumpu za beton, radnog učinka 150 m³/h, sa dužinom strele od 36 m.



Kao generalni izvođač radova, vršimo koordinaciju svih učesnika na projektu, planiranje, praćenje i nabavku materijala, kontrolu kvaliteta izvedenih radova, poštujući zadate vremenske rokove i finansijski okvir investitora.



Osnovi princip našeg poslovanja zasniva se na individualnom pristupu svakom klijentu i pronalaženje najoptimalnijeg rešenja za njegove transportne i logističke potrebe.



Usluge građevinske mehanizacije vršimo tehnički ispravnim mašinama, sa potrebnim sertifikatima kako za rukovođe građevinskim mašinama tako i za same mašine.



Raspoložemo opremom i mašinama za sve zemljane radove, kipe i dampere za rad u teškim terenskim uslovima, automiksere i pumpe za beton, autodizalice, podizne platforme.



Sakupljanje i privremeno skladištenje otpada vršimo našim specijalizovanim vozilima i deponujemo na našu lokaciju sa odgovarajućom dozvolom. Kapacitet mašine je 250 t/h građevinskog neopasnog otpada.



NIŠ

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MOBECO

GRADIMO SADAŠNJOST ZA ODRŽIVU BUDUĆNOST



Strastvenost, timski duh i vizija budućnosti. Mapei aktivno doprinosi najvažnijim svetskim projektima u oblasti arhitekture, infrastrukture i stanovanja, kao i obnovi kulturno-istorijskih građevina. S predanošću radimo svakog dana, kako bismo oblikovali održivu budućnost građevinske industrije.



Više na: www.mapei.rs i www.mapei.com

MATEST "IT TECH" KONTROLNA JEDINICA



JEDNA TEHNOLOGIJA MNOGO REŠENJA

IT Touch Technology je Matestov najnoviji koncept koji ima za cilj da ponudi inovativna i user-friendly tehnologiju za kontrolu i upravljanje najmodernijom opremom u domenu testiranja građevinskih materijala

Ova tehnologija je srž Matestove kontrolne jedinice, software baziran na Windows platformi i touch screen sistem koji je modularan, fleksibilan i obavlja mnoge opcije

- | | |
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SISTEM JEDNOG RAZMIŠLJANJA JEDNOM SHVATIŠ - SVE TESTIRAŠ



NAPREDNA TEHNOLOGIJA ISPITIVANJA ASFALTA

- | GYROTRONIC - Gyrotory Compactor
- | ARC - Electromechanical Asphalt Roller Compactor
- | ASC - Asphalt Shear Box Compactor
- | SMARTTRACKER™ - Multiwheels Hamburg Wheel Tracker, DRY + WET test environment
- | SOFTMATIC - Automatic Digital Ring & Ball Apparatus
- | Ductilometers with data acquisition system

MULTIFUNKCIONALNI RAMOVI ZA TESTIRANJE

- | CBR/Marshall digital machines
- | Universal multispeed load frames
- | UNITRONIC 50kN or 200kN Universal multipurpose compression/flexural and tensile frames

OPREMA ZA GEOMEHANIČKO ISPITIVANJE

- | EDOTRONIC - Automatic Consolidation Apparatus
- | SHEARLAB - AUTOSHEARLAB - SHEARTRONIC
- Direct / Residual shear testing systems
- | Triaxial Load Frame 50kN

MIXMATIC - Automatic Programmable Mortar Mixer