



PACE-2026

The Third International Congress on the
Phenomenological Aspects of Civil Engineering

Atatürk University, Erzurum, TÜRKİYE

17-19 September 2026

Transactions of the PACE-2026

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Edited by Abdulkadir Cüneyt Aydın

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Foreword

Civil engineering is as old as mankind, and, which will be forever. The civilization, which includes all applied sciences is the basics of civil engineering. Thus, the gap, between pure science and enhancing quality of life is the wisdom of civil engineering, which is a combination of art, craft, and mathematics. Thereby, the Congress name should imply the scientific era, which is inspected intensively as long as mankind. That means also, many things have done before. This concept, gathered us to decide a name, which can be memorized and remembered easily, harmonic, and emphasized the openness of the Congress. Withal, the abbreviation of the Congress has to be a meaningful word. For a pandemic world, to activate the scientific impression to our lives, International Congress on Phenomenological Aspects of Civil Engineering (PACE) get in life.

As the above-mentioned sense, PACE-2026 aimed to get in touch the academicians, researchers and practitioners in the civil engineering, geological engineering, geophysical engineering, architectural engineering, architecture, and landscape architecture related topics from all around the world to review recent trends, topics, problems, and innovations of knowledge and understanding in the relevant areas to share the knowledge that the present, and the future pose. It is hoped that the transactions of the PACE-2026 will provide a fruitful background for up grading the branch, through an appropriate exchange of information between all researchers, scientists and industrial scholars.

The impression of a congress is very important, especially when it is executed for the third time. Thus, 2 keynotes from India and Türkiye are selected. A total of 17 manuscripts from 9 countries are submitted. All authors submitted their full manuscripts, except for one who submitted their work as an abstract. Withal, only the presented submissions are included within this transactions e-book. The reviews of the papers were undertaken by the Scientific Committee of the congress, and only the accepted ones were announced. The editor is deeply grateful to all for their effort in enhancing the quality of the transactions. At the end, the PACE-2026 Congress is executed with 2 keynotes, 17 presentations, and 28 participants from 9 countries, ranging from Algeria to China, Belgium to Indonesia, Greece to Malaysia, and Kosovo to Iran. Hereby, about 64.7% of the manuscripts were from outside of Türkiye, which implies the globality of the Congress. We think, the Congress got the pace...!

I would like to thank to all participants of the congress, especially the session chair and Keynote from; to get in touch at PACE-2026, At the second stage, all PACE-2026 Committee members, they have done their best to get in life this Congress. Without their grateful support, the PACE-2026 would not have been the great success it was.

We look forward to get in touch again in the next congress, PACE-2028.

Abdulkadir Cüneyt Aydın

Editor & Chairman of the PACE-2026

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KEYNOTE SPEAKERS



Technical and Economic Controversies in Earthquake Engineering: A Comparative Evaluation of Seismic Isolation and Structural Health Monitoring

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Keywords

*Seismic Isolation,
Structural Health
Monitoring,
Earthquake Engineering,
Performance-Based Design,
Structural Resilience.*

Abstract

This paper explores the technical and economic debates over implementing seismic isolation and structural health monitoring in modern earthquake engineering. As the field moves from a focus on life-safety to performance-based systems prioritizing functional recovery, seismic isolation offers a shift by separating structures from ground motion. However, the study reveals significant technical hurdles, such as vulnerability to near-fault velocity pulses that may cause structural pounding, and the long-term degradation of elastomeric bearings from environmental stressors. Additionally, environmental noise and hardware attrition can undermine the reliability of structural health monitoring, creating a gap between data collection and actionable intelligence that prevents proactive maintenance. An examination of current design codes, like ASCE 7, demonstrates how prescriptive regulatory minimums and conservative property modification factors can result in suboptimal designs. The paper contends that bridging this implementation gap requires moving toward risk-informed, hazard-consistent approaches that incorporate real-time sensor data into regulatory standards. Ultimately, the results suggest that while high-performance systems come with a higher initial investment, their adoption is economically justified by reducing lifecycle costs and preventing functional downtime, provided that engineering practices shift from mere code compliance toward holistic, data-driven resilience.

1. Introduction: The Resilience Gap in Modern Earthquake Engineering

Whether a hospital or other strategic structure remains operational after an earthquake is now a question of performance, not just of structural strength. Modern engineering has evolved from life-safety objectives to performance-based criteria to ensure that structures stay open for operation [1]. Conventional strengthening can unintentionally result in larger inertial forces being exerted on a building, while seismic isolation (SI) works by decoupling the structure from the shaking ground [2]. The importance of SI is made evident by the more than 23,000 structures worldwide that utilize passive anti-seismic devices for protecting military and medical centers [3]. As Figure 1 shows, these techniques involve mechanical decoupling and information-based measures, enabling a structure to remain fully functional. This change from mere code compliance to cost-risk analysis will allow a structure to remain fully functional following a major earthquake [1].

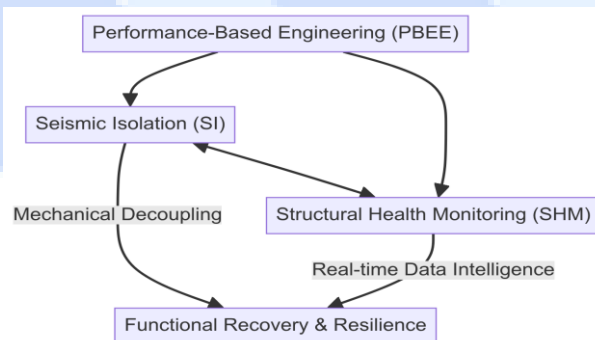


Figure 1. Conceptual Framework of Seismic Resilience

However, high-performance engineering is subject to both technical and economic controversies. Research has identified an “ugly” side of SI, which includes the physical vulnerabilities of these structures to near-fault pulses and spectral anomalies that can trigger structural pounding [4]. Further, SHM has its own problems, as instrument aging and environmental noise can lead to poor decision-making when it comes to maintenance [5]. These technical issues are made even worse by the high cost of SI, with an estimated retrofit cost between 200 and 500 USD/m², which is much higher than conventional methods [2]. The comparison between prescriptive code safety and advanced resilience systems is explored in Table 1. Furthermore, physical mechanisms like sensor bond breakdown and thermal stress are causing problems for SHM, hindering the development of an effective real-time monitoring system [5].

Table 1. Comparison of Traditional vs. High-Performance Seismic Strategies

Feature	Prescriptive Design (Code Minimum)	Seismic Isolation & SHM
Primary Goal	Life Safety / Collapse Prevention	Functional Recovery / Resilience
Structural Response	Inelastic deformation (Ductility)	Rigid body motion (Decoupling)
Initial Cost	Low (\$50–150/m ²)	High (\$200–500/m ²)
Post-Event State	Significant repair / Downtime	Immediate occupancy / Operational
Maintenance	Visual inspections only	Continuous sensor-based monitoring

In this work, I investigate the effect of technical uncertainties and economic trade-offs on the long-term reliability of high-performance infrastructure. One of the main challenges for SI in the U.S. is the prescriptive code that limits the number of buildings utilizing this technology to approximately 250, compared to thousands of buildings using this technology in Japan [3]. The prescriptive code only provides a basic level of safety and fails to address the environmental degradation of structures over time and the near-fault earthquake demands that will impact performance [3]. Therefore, I investigate how real-time data from sensors can be used in risk-informed regulatory standards to reduce the current implementation gap. I am investigating the integration of mechanical isolation and digital monitoring to improve the resilience of critical infrastructure networks in the U.S. [1]. I will establish the theoretical metrics for success before discussing technical disputes and regulatory shortcomings.

2. Theoretical Framework: PBEE and Reliability Theory

To advance from code-minimum life safety to functional recovery, we must create a reliable connection between engineering requirements and structural capabilities. This chapter details the metrics needed to assess seismic isolation technologies, moving past simple safety margins and toward explicit goals for performance. The sections that follow will give a brief overview of Performance-Based Earthquake Engineering principles in Section 2.1, along with the mathematical way we account for aleatory and epistemic uncertainty in Section 2.2.

2.1. Principles of Performance-Based Engineering: From Strength to Deformation

Modern seismic engineering is moving away from a strength-based design to a performance-oriented framework, because the two parameters have historically been mistakenly considered as synonyms, leading to variable risk outcomes among different structures [6]. While traditional design estimates internal forces, Performance-Based Earthquake Engineering (PBEE) is concerned with managing structural damage by controlling deformations [7]. This entails a shift from the empirical force reduction factors to checking inelastic deformations and material strains in the members or sections of the structure, as is demonstrated in Table 2 [6, 7]. Consequently, the quantitative assessment of structural behavior moves beyond assumption-based qualitative analysis toward science-based predictive modeling [7]. This evolution is vital for ensuring high-performance systems like seismic isolation are evaluated based on their ability to control physical damage rather than just resisting lateral forces.

Table 2. Comparison of Strength-Based vs. Performance-Based Design

Feature	Strength-Based Design	Performance-Based Design
Primary Focus	Internal forces and strength [7]	Deformations and damage [7]
Analysis Type	Linear elastic (mostly)	Nonlinear (static or dynamic) [8]
Safety Metric	Force reduction factors (R)	Reliability indices and limit states [9]
Verification	Implicit (assumed)	Explicit (verified at local levels) [7]
Goal	Life safety minimums	Multi-level resilience targets [10]

The essence of PBEE is to link desired performance levels with earthquake return periods, rather than a single-level life-safety minimum required by older building codes [10]. Engineering limit states, like Immediate Occupancy or Life Safety, are translated into physical parameters, such as interstory drift, plastic rotation, and energy dissipation capacity [7, 10]. In this context, damping and period lengthening are key tools in protecting the superstructure, as demonstrated through displacement-period spectra analysis [6, 11]. Adaptive systems improve this further by adjusting the stiffness and damping of the structure based on the earthquake loading intensity [12, 13]. This displacement-based approach allows for an assessment of the reliability of isolated and monitored infrastructure as addressed in this work.

2.2. Quantifying Uncertainty in Structural Systems: Aleatory Randomness and Epistemic Bias

The discipline of seismic risk analysis establishes a basic distinction between aleatory and epistemic uncertainty. Aleatory uncertainty represents the inherent randomness of repeatable experiments. Epistemic uncertainty refers to uncertainty due to lack of knowledge [9]. This distinction is critical for understanding structural risk, as Figure 2 indicates. Aleatory randomness cannot be reduced. Epistemic uncertainty is reduced as more samples are available, vanishing as the sample size increases to infinity [14]. The boundary between aleatory and epistemic uncertainty is model-dependent. If the randomness is captured by a stochastic model, it is categorized as aleatory. The uncertainty in the parameters of this stochastic model is epistemic [9]. Thus, structural reliability is a statement of the quality of information. As information is added, the total uncertainty is reduced [14]. This concept is vital for this study, as it relates reliability to our knowledge of high-performance systems.

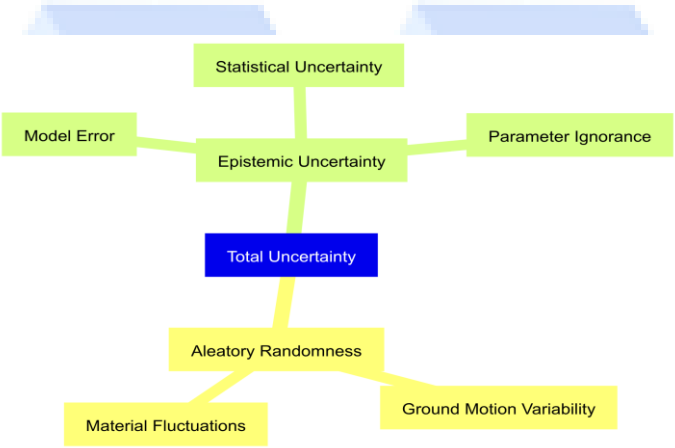


Figure 2. Classification of Uncertainties in Seismic Engineering

The First-Order Reliability Method (FORM) is used to map structural demand against capacity by approximating the limit state surface with a tangent hyperplane at the most likely failure point [9, 14]. In a normalized Gaussian space, the reliability index (β) represents the formulation-invariant shortest distance from the origin to this failure plane [14]. The deterministic model makes assumptions about geometry and boundary conditions. As a result, model error exists. This model error is aleatory and cannot be removed. However, there is also epistemic uncertainty due to unknown geometric and boundary factors. To account for this, judgmental variables are included in the model [14, 16, 17]. The Bayesian statistical approach is a useful tool to estimate these judgmental variables. It allows for the use of expert judgment and qualitative information to quantify structural plausibility with limited information [9, 16]. By treating probability as a multi-valued logic of plausibility, engineers can investigate how much residual uncertainty remains when using an optimal deterministic model for response predictions [16].

3. Technical Controversies in Seismic Isolation

Seismic isolation is often seen as the best way to protect infrastructure, but it creates significant technical conflict zones when dealing with extreme seismic conditions. This chapter examines the gap between how things are supposed to work mechanically and the shortcomings of modern structural engineering codes. The evaluation below covers near-fault pulse vulnerabilities in Section 3.1, elastomer aging mechanisms in Section 3.2, and the prescriptive paradoxes of ASCE 7 standards in Section 3.3.

3.1. Near-Fault Pulse Effects and Pounding Risks in Seismic Isolation

Conventional seismic isolation typically works by shifting a structure's natural period into a region of lower seismic energy, often under the assumption of a idealized spectral shape [4]. But ground motions near faults are frequently characterized by long-period velocity pulses caused by forward-directivity effects, which can significantly reduce these period-lengthening advantages [18, 19]. As Table 3 indicates, these pulse-like excitations generate significant displacement demands, which may go up to 1.0 meters and surpass the failure capacity of traditional elastomeric bearings [18]. Furthermore, linear isolation systems may exacerbate high-frequency vibrations in the superstructure, particularly if the seismic energy is concentrated around the isolated period [19]. Consequently, the presumed safety margins of passive isolation systems are often compromised in near-fault regions, where the seismic characteristics do not align with standard design assumptions.

Table 3. Comparison of Near-Fault and Far-Field Isolation Performance

Performance Metric	Far-Field Excitation	Near-Fault (Pulse-Like) Excitation
Base Shear Reduction	High (up to 50% reduction)	Variable; potential for amplification
Displacement Demand	Stable; typically, within design limits	High (can exceed 1.0 m)
Spectral Energy	Distributed across frequencies	Concentrated in long-period pulses
Superstructure Yielding	Minimal to none	Significant (30-60% of elements)
Pounding Risk	Low	High (exceeding rattle space)

Numerical time-history analyses indicate that isolated structures may undergo greater roof displacements than fixed-base buildings when exposed to forward-directivity pulses [4, p. 10]. Near fault lines, spectral bumps at the isolated period can invalidate low-damage design goals, with between 30% and 60% of structural elements expected to yield [4]. Conventional intensity measures like Peak Ground Acceleration (PGA) are poor predictors of this response, as isolator displacement is primarily velocity-dependent and governed by Peak Ground Velocity [20, 21]. Therefore, the Effective Incremental Ground Velocity (EIGV) is considered a better indicator of isolation demand, as it accounts for both the pulse duration and acceleration in relation to the system's friction strength [21]. Transitioning to velocity-based metrics is vital for guaranteeing the performance of critical infrastructure under the non-linear demands of near-fault ground motions.

When the design clearance or rattle space is exceeded, shock waves travel up the height of the building, often leading to peak floor accelerations above 2.5g and inter-story drifts over 3% [4]. Pounding against moat walls causes severe plastic deformation and increases the risk of building rollover or bearing uplift, owing to elevated accelerations [22]. Mitigation involves a critical balance; for instance, increasing isolator yield strength and nonlinearity can control excessive displacement, but it may lead to higher base shear [18, 20]. As shown in Figure 3, advanced technologies such as Nonlinear Inertial Amplifiers (NIARRI) offer a compelling solution, delivering an average displacement reduction of 32.48% across various seismic records and preventing pounding [23]. The results suggest that shifting from passive to nonlinear damping is crucial for ensuring structural integrity near faults.

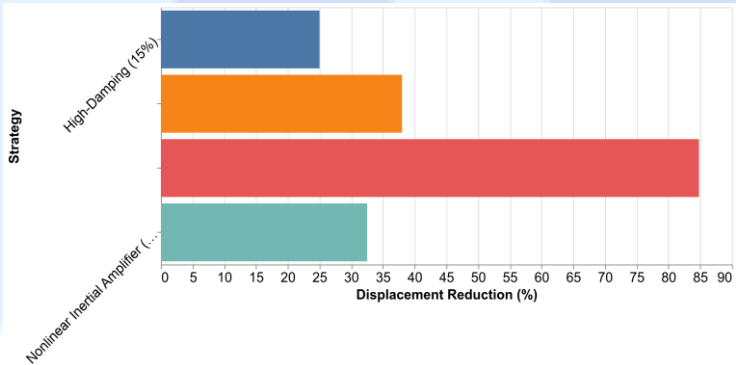


Figure 3. Displacement Reduction via Nonlinear Mitigation Strategies

Description: Effectiveness of advanced mitigation techniques in reducing peak isolator displacement under near-fault loading.

Note: NIARRI refers to Nonlinear Inertial Amplifiers. Yield strength increase refers to shifting from 6% to 12% deck weight.

3.2. Mechanisms of Elastomer Degradation and the Property Modification Controversy

The deterioration of high-damping rubber bearings over long periods is mainly attributed to chemical transformations such as chain scission and changes in crosslink density [24]. As indicated in Figure 4, these alterations result from extended vulcanization and continued exposure to environmental factors like ozone and oxygen [25]. The resulting changes are an increase in horizontal stiffness and a decrease in energy dissipation capacity [24]. Mechanical degradation also entails the initiation of microcracks, where internal flaws expand under high-displacement cyclic loading [24]. In intense seismic events, this crack growth can eventually result in cavitation failure or shear separation of the rubber layers from the steel reinforcement [26]. Understanding the underlying mechanisms is essential for estimating the long-term reliability of isolated structures for their intended 50-year service life.

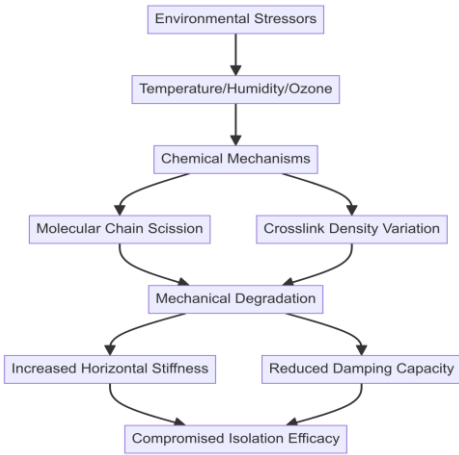


Figure 4. Elastomeric Bearing Aging and Degradation Pathways

Description: Chemical and mechanical mechanisms leading to long-term stiffness evolution.

Note: Environmental coupling (Temp/Ozone) accelerates chemical degradation. Stiffness recovery (Mullins effect) occurs over a 5-year cycle.

The synergistic effect of multiple environmental factors, such as thermal-moisture cycles and salt spray, causes the bearing degradation to accelerate significantly compared to single stressors [24]. The mechanical behavior of high-damping rubber bearings (HDRB) is affected by temperature, axial load, and the Mullins effect [26]. Experimental dynamic testing shows that higher strain rates or velocities result in higher equivalent viscous damping and effective stiffness [25]. In lead-rubber and sliding bearings, heat generated during cyclic loading can cause a reduction in the friction coefficient and lead core stiffness, making it difficult to predict performance deterministically [26]. This indicates that property modification factors need to account for the coupled effect of material compounding and velocity conditions, rather than considering aging as a single variable. This suggests that deterministic values can be unreliable in the preliminary design of high-performance earthquake engineering.

While the Mullins effect causes temporary softening or scragging, it has been shown experimentally that virgin stiffness properties are largely recovered within five years, with 65 percent of the recovery taking place in the first year [25]. For this reason, designs should be based on the unscragged virgin properties, as ignoring the recovery can lead to an underestimate of the maximum forces transferred to the substructure [25]. Current aging factors like the AASHTO values of 1.2 to 1.3 appear conservative, given the five-year empirical data showing aging factors between 1.0 and 1.2 [25]. However, with the rise of Performance-Based Earthquake Engineering (PBEE), static prescriptive codes may still fail to account for long-term probabilistic behavior and cumulative damage [24]. While safety factors are adequate, this work concludes that there is a need for risk-informed lifecycle assessment to accurately predict the long-term reliability of smart isolation systems in complex environments.

3.3. Prescriptive Limits and Structural Paradoxes in ASCE 7 Seismic Isolation Standards

The current standards establish a specialized regulatory framework that mandates bounding analysis for isolator property uncertainties [27]. As Figure 5 demonstrates, the analysis must consider upper and lower bound properties to identify the maximum possible forces and displacements, accounting for factors such as aging, contamination, and operating temperature [28]. The property modification factors, or lambda factors, are employed to account for uncertainties arising from operational heating and limited experimental data on actual isolators [28, 29]. The default 'unknown' manufacturer factors are often over-conservative compared to tested systems [28]. The prescriptive bounds used in current standards represent the primary approach for ensuring long-term reliability in the face of environmental degradation and manufacturing tolerances.

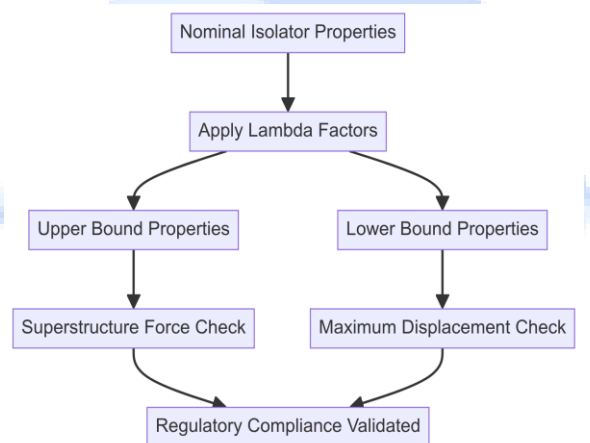


Figure 5. ASCE 7 Bounding Analysis Procedure

Description: Regulatory workflow for evaluating isolator property uncertainties.

Note: Upper bound properties govern force demands on the superstructure. Lower bound properties govern maximum displacement demands.

The standards have increased the minimum number of ground motion records to 11 to reduce statistical scatter in response history analysis compared to the 7-record suites used in earlier versions of the U.S. standards [27, 30]. Prescriptive scaling often uses the Uniform Hazard Spectrum, which may envelope distinct earthquake scenarios and fail to represent realistic physical events [28, 30]. Regulatory floors, such as minimum upper bound periods for short-period buildings, can also lead to over-conservative design demands [30]. Probabilistic assessments show that scaling methods can violate hazard consistency, indicating that reliability-based assessments provide a more accurate estimate of structural demand than the simple scaling methods allowed by ASCE 7 [29]. This study identifies a critical need for code evolution to better reflect the true dispersion of structural responses through hazard-consistent methodologies.

4. Reliability and Data Interpretation in SHM

Even though seismic isolation systems handle a structure's mechanical strength during earthquakes, keeping high-performance infrastructure reliable for the long haul requires moving beyond periodic visual inspections to constant monitoring with sensors. This marks a major shift in how we track structural health, but it also creates a gap between raw data and real insights, where raw numbers need to be turned into trustworthy diagnostic signals. The big hurdle is sorting out real structural issues from all the environmental noise, operational changes, and the slow wear and tear of the sensing equipment. This chapter looks at these main obstacles to reliability. It starts with Section 4.1, which looks at how environmental variability affects data and the statistical tricks to cut down on false alarms. After that, Section 4.2 looks at how long hardware actually lasts and if it's worth it financially to do proactive maintenance. Together, they give you a solid way to judge if your monitoring data is actually useful.

4.1. Environmental Noise and Statistical False Alarm Mitigation in SHM

Damage indices (DIs) in vibration-based structural health monitoring are intrinsically stochastic due to the influence of environmental factors like temperature and humidity [31]. Operational variability, such as wave-induced excitations or tidal effects on bridge resonance, results in modal frequency changes that mimic structural damage [32]. Additionally, high-frequency sensor noise and sampling variability make filtering and preprocessing essential for a stable baseline [33, 34]. Failure to disentangle reversible environmental cycles from irreversible structural degradation causes false positives [31, 35]. A robust baseline requires understanding how ambient conditions impact the dynamic characteristics of infrastructure, as signal drift can cause major issues in the long-term.

Automated alerts are prone to the base rate fallacy, where even highly accurate sensors can result in inaccurate diagnoses if the prior probability of damage is low [36]. As Figure 6 shows, transitioning to a two-stage monitoring approach using binomial distribution classifiers allows engineers to filter transient spikes by tracking threshold crossings over temporal windows [31, 37]. Additionally, Mutual Information Independence (MII) and Kalman filtering provide decentralized ways to identify and reconstruct signals from malfunctioning or debonding sensors [38]. Creating weighted data quality indicators (DQIs) ensures that maintenance decisions are based on the correctness and timeliness of the sensor network rather than raw, potentially corrupted inputs [37]. These statistical frameworks offer a mathematically rigorous basis for structural health decisions, mitigating measurement uncertainty.

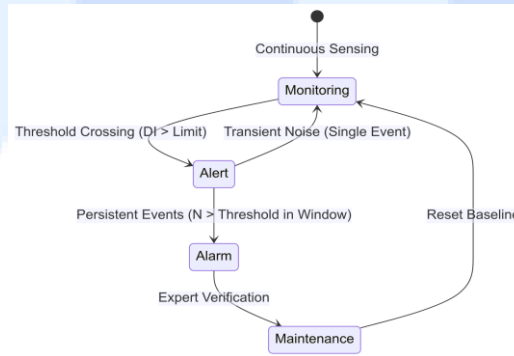


Figure 6. Binomial Distribution Classifier for False Alarm Mitigation

Description: Two-stage statistical process to differentiate between transient noise (Alert) and persistent structural damage (Alarm).

Note: The system uses a temporal observation window to filter out spurious threshold crossings. An 'Alarm' is only triggered if the number of 'Alerts' exceeds a statistical probability threshold.

Automated clustering algorithms, such as DBSCAN and TK-means, increase reliability by separating physical resonance modes from computational artifacts and noise [33, 39]. While deep learning models like LSTMs can accurately predict responses, they struggle with black-box opacity and event timing delays [34, 40]. Explainable AI (XAI) frameworks are necessary to map abstract algorithmic features to physical quantities like eigenfrequencies to build engineering trust [40, 41]. Furthermore, human-in-the-loop (HITL) systems are still necessary to prevent crying wolf, where excess false alarms cause the disregard of valid structural warnings [36]. Combining domain knowledge with automated systems ensures that algorithms do more than just identify patterns, but offer physically meaningful insights to overcome the limitations of data-driven models.

4.2. Hardware Longevity and Maintenance Cycles in Structural Health Monitoring

In harsh infrastructure settings, SHM reliability is frequently impacted by hardware wear and tear. Long-term case studies on bridge substructures reveal high failure rates, with approximately 27% of sensors becoming non-functional after a 20-year service life [42]. As Figure 7 demonstrates, the primary culprit for gauge deterioration, particularly for those submerged below groundwater, is moisture ingress [42]. Specialized material coatings, such as polyimide for FBG fibers and hydrophobic epoxy for PZT transducers, are employed to enhance durability [43]. Regardless of these safeguards, environmental and operational cycles cause noticeable signal drift of 0.01–0.02% per day, necessitating physical recalibration every 4 to 6 months to ensure data integrity [44]. These findings highlight that sensor survivability is a major challenge to the reliability of long-term monitoring.

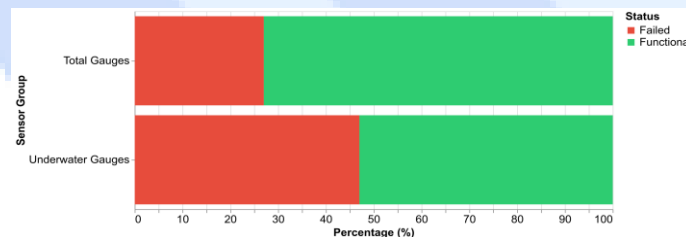


Figure 7. Long-Term Sensor Failure Rates in Bridge Substructures

Description: Empirical failure rates of sensors observed over a 20-year service life in harsh structural environments.

Note: Submerged/underwater sensors exhibit significantly higher failure rates (approx. 47%) compared to the general population. Overall failure rate across all gauges reached 27% at the 20-year mark.

Stochastic degradation models, such as Petri nets, which model hardware wear-out using Weibull distribution parameters, are increasingly used to inform effective maintenance cycles [45]. Information theory metrics, like Expected Information Gain (EIG), provide a way for engineers to quantify how sensor failure impacts the system's ability to reduce uncertainty regarding structural damage [45]. Data quality in wireless platforms is highly unpredictable, with average data loss rates of 30%–50% resulting from transmission barriers [46]. Network topology is also critical, as the failure of a single strategically placed sensor often causes a greater increase in monitoring error than the loss of multiple redundant nodes [45]. This mathematical framework illustrates that reliability is spatially sensitive, meaning sensor placement must be optimized to ensure that hardware loss does not undermine the monitoring network's diagnostic capability.

Early detection of internal structural decay at approximately 40% of ultimate strength, before surface cracks appear, is made possible through proactive monitoring-on-demand [43]. Life Cycle Cost Analysis (LCCA) indicates that SHM can increase bridge service life from 40 to 65 years, thereby improving the benefit-to-cost ratio by 17.5% [47]. However, the economic impact of maintenance includes significant costs associated with specialized labor and training, as stakeholders deal with productivity losses during the learning phase [46, p. 99]. Operational costs for maintaining the sensor network are typically around 10% of the initial capital investment each year [47]. In this work, these findings indicate that although SHM requires consistent technical and financial support, the extended service life provides strong economic justification for shifting away from reactive repairs to proactive, sensor-based infrastructure management.

5. Synthesis: Economic Realities and Code Harmonization

High-performance infrastructure will only be viable if technical resilience is balanced with economic sustainability. While seismic isolation and structural health monitoring have been proven to have mechanical and data-driven capabilities in previous chapters, their adoption is often limited by their high initial costs. This chapter synthesizes these findings and demonstrates why life-cycle value and real-time data are the most important factors for adopting smart systems in modern engineering. By exploring the transition from voluntary recommendations to data-driven mandatory standards, this chapter examines life-cycle cost-benefit analysis in Section 5.1 and the movement toward regulatory harmonization and mandates in Section 5.2.

5.1. Lifecycle Cost-Benefit Analysis of Integrated Seismic Systems

The uptake of seismic isolation (SI) and structural health monitoring (SHM) is more limited by the high upfront capital cost than by the technologies themselves [48]. An SHM system on a large structure generally costs roughly 3% of the rebuild value [49], and major bridge projects can run \$1 million for the instrumentation alone [47]. Nevertheless, the 883 USD/m² price tag for seismic isolation can be offset by a smaller superstructure, such as reduced braced frames [48]. A lower-cost alternative, like fiber-reinforced recycled-rubber bearings, can reduce the manufacturing cost by 40% to 60% compared to lead-core systems [50]. The technical-environmental feasibility of circular economy strategies [50] means that the initial investment required for high-performance earthquake engineering is decreasing.

According to Lifecycle Cost Analysis (LCCA), monitoring saves money by increasing the operational life of the infrastructure from 40 years to 65 years [47]. With CBM, the benefit-to-cost ratio increases by approximately 17.5% [47], as illustrated in Table 4. Thus, by using the information from SHM, the expected annual losses of isolated structures are often half or less than the losses of non-isolated structures [48]. The value of SHM is also evident in the prevention of functional downtime, which can cost society more than \$113,000 a day [47]. By managing infrastructure using a risk-informed approach, total risks and operational costs can be reduced by as much as 90% [51]. In summary, while standard discount rates may suggest that we should pursue myopic policies [52], the dynamic simulation of out-of-equilibrium recovery benefits suggests that the increased upfront costs of monitoring and isolation are well-justified [52].

Table 4. Economic Comparison: Proactive SHM vs. Reactive Maintenance

Metric	Reactive Approach (Baseline)	Proactive SHM-SI Approach
Service Life Assumption	40 Years [47]	65+ Years [47, 53]
Annual Economic Benefit	Baseline	~\$41.2M (Loss Avoidance) [47]
Benefit-to-Cost Ratio	4.79 [47]	5.63 (17.5% increase) [47]
Maintenance Strategy	Schedule-based / Corrective	Condition-Based (CBM) [54]
Reliability Confidence	Low (Visual/Ad-hoc)	High (Continuous/Real-time) [54]

Description: Comparison of financial and operational metrics between traditional reactive maintenance and SHM-facilitated proactive strategies based on bridge case studies.

Note: SHM-equipped structures show higher initial costs but significantly higher benefit-to-cost ratios. Service life extension is a primary driver of long-term economic superiority.

5.2. Integrating Monitoring and Isolation into Regulatory Standards: From Recommendations to Mandates

In 2016, a regulatory breakthrough for structural health monitoring (SHM) occurred when the Chinese Ministry of Transportation published the first national code that legally mandated sensor installation on major highway bridges [55]. This differs from the common practice in the West, where codes like those from the FHWA in the United States or SAMCO in Europe function as voluntary technical recommendations instead of binding legal obligations [55]. As Figure 8 demonstrates, incorporating SHM into design codes as a requirement involves determining the right applicability thresholds, such as bridge span length, that trigger legal enforcement [55]. This shift requires a change in how safety responsibility is perceived, shifting from periodic manual inspections to relying on permanent in-situ installations and automated diagnostics [56]. This move

towards establishing enforceable technical standards is vital for closing the existing implementation gap and ensuring that monitoring systems remain as sustainable as the high-performance infrastructure they protect.

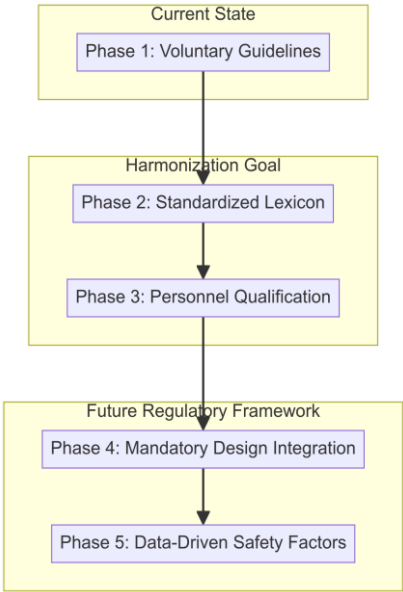


Figure 8. Regulatory Roadmap for SHM-Code Integration

Description: Proposed transition from voluntary technical recommendations to mandatory, data-driven regulatory standards.

Note: The roadmap leverages lessons from the Chinese mandatory SHM code. Standardization of personnel qualification is a prerequisite for industrial recognition.

Future seismic codes might benefit greatly from moving towards risk-targeted design, where partial safety factors are modified based on sensor-verified structural damage levels [57]. Real-time data analysis allows for the continuous update of structural degradation models, which can replace static code-based property modification factors with site-specific deterioration rates measured directly [54]. To successfully achieve this, a universal “grammar” or vocabulary for data interpretation needs to be created to minimize diagnostic ambiguity and ensure regulatory compliance across different disciplines [58]. By integrating structural material characteristics and code-defined stress limits directly into automated evaluation loops, the engineering community can move from reactive verification to proactive, data-driven design [59]. This study argues that this level of harmonization is necessary to use structural health monitoring data to support reduced safety factors while still maintaining uniform reliability targets across different seismic environments.

6. Conclusion: Navigating the Technical and Economic Future of Seismic Engineering

This work evaluates the critical gap between theory and practice in the design of isolated structures. The protection of isolated structures is limited by the “ugly” side of isolation, which includes large displacement demands in near-fault zones [60]. Numerical studies have indicated that structures designed for the minimum criteria of ASCE 7-10/16 exhibit unacceptably high collapse probabilities, often exceeding 10%, which leads to the paradoxical conclusion that they are less reliable than conventional structures during extreme events (Table 5). In addition, an increase in the building strength can sometimes lead to increased failure probabilities [29]. This risk is compounded by the superstructure strength paradox, where increasing building strength without proportionate isolator capacity can elevate failure probabilities [29]. To achieve acceptable levels of reliability, engineers should specify a response modification factor of 1.0 and stiffening isolators with a capacity that is at least 1.5 times the average demand [29]. To achieve these goals, design must move beyond prescriptive minimums toward system-specific design strategies that explicitly coordinate isolator limits with superstructure behavior [29].

Table 5. Comparison of Collapse Risks: Isolated vs. Non-Isolated Structures

Design Configuration	Collapse Probability (MCER)	Key Vulnerability Factor
Minimum Code Isolated SMF	~40% (without restraint)	Near-fault pulse & pounding
Minimum Code Non-Isolated	< 10%	Inelastic superstructure damage
Enhanced Isolated (RI=1.0)	< 5%	Displacement capacity (1.5x demand)
Isolated with Stiffening	~6.1%	Stiffening restrainer ring effectiveness

Description: Summary of collapse probabilities under Maximum Considered Earthquake (MCER) conditions based on minimum code design criteria.

Note: Minimum code compliance for isolated structures can lead to higher collapse risks than fixed-base counterparts due to displacement limits. Stiffening behavior in isolators is a critical factor in preventing total system failure.

The gap between theoretical models and real-world structural integrity can be bridged by structural health monitoring (SHM), allowing a transition from static maintenance to data-driven post-event functionality assessments [60]. The reliability of these digital networks is often hindered by unmodeled interactions, such as masonry infill effects, which significantly alter intended load paths and dynamic characteristics [61]. Emerging self-centering rocking systems provide a more predictable baseline for these assessments, simplifying the task for SHM networks to distinguish actual structural distress from environmental noise [61]. Despite these advancements, this study identifies that current SHM integration remains limited by a lack of empirical data on long-term bearing aging and the mechanical complexity of newer 3D protection systems [60]. Addressing these gaps requires a fundamental shift in how monitoring data is used to validate structural health, ensuring that sensors provide actionable intelligence rather than just raw measurements in the face of environmental degradation and complex structural geometries.

The future of earthquake engineering requires a paradigm shift from life-safety minimums toward functional recovery, integrating ancient mechanical wisdom like rocking blocks with modern sensor technology [61]. As Figure 9 illustrates, the synergy between mechanical decoupling and digital sensing is vital for achieving resilience-based design targets. Future regulatory codes must evolve toward system-dependent requirements that link isolator characteristics directly to the specific seismic force-resisting system [29]. Economic controversies, often cited as a barrier to adoption, can be mitigated through lifecycle assessments that justify high initial costs by reducing insurance premiums and minimizing economic downtime [60]. A final verdict suggests that while current prescriptive codes like ASCE 7 are insufficient for extreme events, the integration of real-time monitoring data offers the only viable path to dynamic, risk-informed infrastructure management [60, 61]. Ultimately, this holistic approach ensures that high-performance infrastructure remains both technically reliable and economically sustainable throughout its service life.

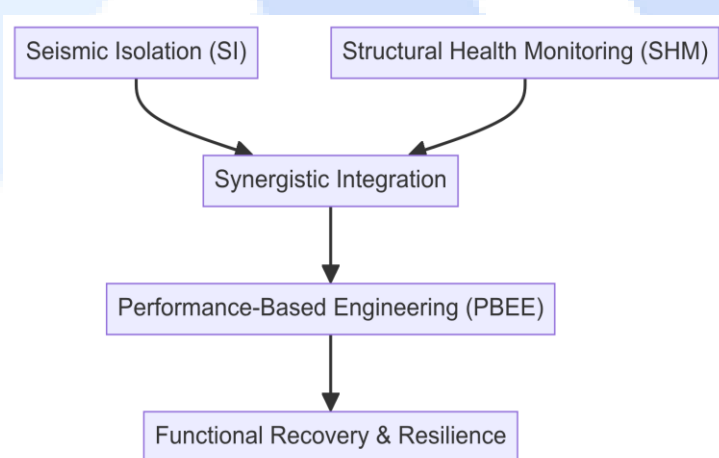


Figure 9. Synthesis of Seismic Resilience: Synergy between SI and SHM

Description: Conceptual framework for the integration of mechanical decoupling and digital sensing to achieve high-performance infrastructure goals.

Note: The synergy aims to bridge the gap between prescriptive code minimums and functional recovery targets. Mechanical decoupling provides the physical protection while digital sensing ensures long-term reliability.

Declaration of Originality

I hereby declare that I have written this paper independently and without external assistance. I have not used any sources or aids other than those indicated. All passages in this work that have been quoted verbatim or paraphrased from publications or other external sources have been identified as such.

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Behaviour of Bridges During Earthquakes: Component-Wise Study

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Keywords

*Seismic pounding,**Ductile detailing,**Capacity design,**Base isolation,**Spherical bearings.*

Abstract

Recent devastating earthquakes have shown that bridges are among the most vulnerable components of transportation systems. It is observed that earthquake damage is mainly due to excessive displacement of the bridge deck and bearings and to the shear failure of piers. Bridges are prone to large bearing displacements and deck accelerations because of their low structural redundancy and small inherent damping. A major earthquake that causes partial or total bridge collapse would seriously hamper relief and rehabilitation operations by disrupting the transportation network. Highway crossings are critical infrastructure as they serve as major access and evacuation routes during and after catastrophic events. Seismic design of bridges is especially important because they are considered lifeline structures. The Northridge earthquake (January 17, 1994) in the United States caused extensive damage and collapse of several conventionally designed highway bridges. Exactly one year later (January 17, 1995), a devastating earthquake struck the city of Kobe, Japan. This event was a massive disaster that overshadowed the Northridge earthquake. Reported damage to highway bridges included damage to approach slabs, abutments, bearings, pile caps, and decks. In both near-field earthquake scenarios, the collapses involved severe column flexural and shear failures due to nonlinear behaviour, and unseating at in-span and abutment bearings due to excessive displacements. Indian earthquakes such as the 1993 Latur, the 1997 Jabalpur, the 1999 Chamoli, and, most recently, the 2001 Bhuj earthquake caused total collapse or severe damage to bridges. This paper discusses component-wise case studies of bridge failure.



ORAL PRESENTATIONS



Numerical Simulation of Seismic Wave Field in The Two-Phase Viscoelastic EDA Media

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Keywords

Two-phase viscoelastic media,

EDA media,

Wave propagation,,

Anisotropy,

Fractured reservoirs.

Abstract

In the present work, wave function formula of two-phase viscoelastic EDA media is established. Then, elastic parameters of two-phase viscoelastic EDA media are obtained. Moreover, wave field simulation of two-phase viscoelastic EDA media is carried out by means of pseudo-spectra method. Apart from fast longitudinal wave and transverse wave, there is also slow longitudinal wave, which can be observed in two-phase viscoelastic EDA media. The amplitude of slow longitudinal wave is larger than those in fast longitudinal wave and transverse wave in the fluid phase whereas it is the opposite case in the solid phase. It can be observed that anisotropy of media makes the wavefront of longitudinal wave and transverse wave deviate from the circle shape in isotropic media. The wavefront shape of transverse wave is more complex than that of longitudinal wave. In addition, there exists wavefront cusp phenomenon. Viscoelasticity of media can cause the drop of seismic wave amplitude. It can be concluded that two-phase viscoelastic EDA media can reflect multiphase, viscoelasticity and anisotropy fairly well. This work will have great significance for investigating fractured reservoirs.



Evaluation of the Use of Nickel Slag as Coarse Aggregate on the Characteristics of Self-Compacting Concrete (SCC)

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Keywords

*Nickel slag,
Self-compacting concrete,
Coarse aggregate,
Compressive strength,
Fresh concrete properties.*

Abstract

The nickel mining industry in Southeast Sulawesi generates quantities of nickel slag as solid waste, the utilization of which remains limited. Therefore, nickel slag has the potential to be utilized as aggregate to support sustainable construction. This study aims to evaluate the use of sourced nickel slag from PT VDNI as coarse aggregate in terms of workability, porosity and density of hardened concrete, and compressive strength of Self-Compacting Concrete (SCC). The evaluation was conducted by comparing the performance of SCC incorporating nickel slag coarse aggregate with a control mixture using Moramo crushed stone (Split Moramo) at three substitution levels (0%, 50% and 100%). SCC workability was evaluated according to EFNARC (2005), including slump flow, V-funnel, and L-box tests, while the porosity and density of hardened concrete were evaluated according to ASTM C642, and compressive strength at 7 and 28 days was tested according to SNI 1974:2011. The test results showed that the slump flow value of SCC containing nickel slag was 650 mm, compared with that of the control mixture at 693 mm; however, all variations remained within the specified range of 550–850 mm. The V-funnel flow time increased from 8 seconds at 0% substitution to 14 seconds at 100% substitution, indicating an increase in viscosity due to the angular particle shape of the nickel slag. The L-box (H2/H1) values ranged from 0.81 to 0.85, with all mixtures meeting the PA2 passing ability class. Regarding the concrete characteristics, concrete containing nickel slag exhibited lower porosity and higher density than the control concrete, consistent with the higher specific gravity of nickel slag (2.77) compared with Moramo crushed stone (2.68). The compressive strength of concrete containing nickel slag was also higher than that of the control concrete, increasing from 22.86 MPa to 27.40 MPa at 7 days and from 26.42 MPa to 31.39 MPa at 28 days, representing an increase of approximately 18.8% compared with the control concrete. The results indicate that nickel slag has the potential to be used as aggregate in SCC, providing a significant improvement in mechanical performance without compromising compliance with the requirements for fresh SCC properties. Self-compacting concrete, coarse aggregate, Compressive strength, concrete properties.

1. Introduction

Indonesia is the leading producer of nickel globally; however, it also faces environmental challenges concerning ex-mining pits, combustion ash waste, nickel slag, and more within the nickel mining sector (nikel.co.id, 2020). At present, the Ministry of Industry, via the research and development unit, is working to enhance the utilization of nickel slag as an industrial raw material that aligns with environmental management in the framework of a sustainable economic initiative [1].

The rapid growth of nickel ore processing in Southeast Sulawesi, largely driven by PT VDNI, has created large amounts of ferronickel slag as an industrial by-product. If left unmanaged, this slag is both a problem and a wasted material. At the same time, natural coarse aggregate supplies in the region, such as the Split Moramo quarry, are under increasing pressure and have variable quality, which raises concerns about the long-term sustainability of normal concrete production. These two situations. An industrial by-product and a strained natural resource. Give a clear reason to look at nickel slag as an alternative coarse aggregate for structural concrete.

Self-Compacting Concrete is a high-performance concrete that can flow under its weight fill formwork and cover reinforcement without mechanical vibration. Its use is decided by rules for filling ability, passing ability and segregation resistance as set by the EFNARC 2005 guidelines. These rules are checked by slump flow, V-funnel and L-box tests [2]. Because Self-Compacting Concrete mixes rely on a balanced aggregate skeleton and paste volume to reach these rules, replacing coarse aggregate with a material that has different specific gravity,

absorption and particle shape. Such as nickel slag. Can change both the fresh and hardened behavior of the concrete. Therefore, systematic evaluation is needed.

Earlier studies have looked at replacing aggregate in concrete with by-products, including nickel slag. Candrika and Zulfikar studied how copper slag can replace aggregate in High-Volume Fly Ash Self-Compacting Concrete (HVFA-SCC). They used 50 % fly ash. Tested copper slag substitutions of 0 % 10 %, 20 % and 30 %. The tests showed that using copper slag up to 20 % could raise the strength of the concrete. At 20 % copper slag substitution the concrete reached a strength of 38.57 MPa after 14 days. These results suggest that copper slag and fly ash, both waste products can improve concrete performance and encourage the use of alternative materials in construction [3].

Destyanto and Wardhono studied how fly ash can replace some of the cement in Self-Compacting Concrete. They tested fly ash contents of 0 percent, 10 percent, 20 percent and 30 percent. The results showed that the amount of fly ash and the amount of superplasticizer changed the workability, the porosity and the compressive strength of the concrete. The best results came with 20 percent fly ash. These results show that fly ash can be used as a replacement for cement. Using fly ash helps make materials more efficient. Using fly ash also helps use waste. Fly ash can be used in Self-Compacting Concrete mixtures [4].

Hijriah et al. (2025) studied how nickel slag could replace part of the aggregate in high-strength concrete. They tried substitution levels of 25 %, 50 % and 75 %. The best result appeared when 50 % of the aggregate was replaced with nickel slag. In that case, the concrete reached a strength of 46.31 MPa, which is a 10.18 % rise over the control concrete that reached 42.03 MPa. This gain is linked to the chemical nature of nickel slag. Nickel slag has a surface that helps aggregate particles lock together, and it contains silica that can boost concrete performance. Thus, nickel slag shows promise as a material that can help make construction more sustainable [5].

Wijaya and Astutiningsih (2020) carried out a literature review on the use of nickel slag as an aggregate in mortar and concrete. The review findings show that nickel slag can be used as a coarse aggregate. Nickel slag can replace up to 100 % of aggregate in concrete. Nickel slag can also replace up to 50 % of aggregate in asphalt mortar applications. Mechanical properties vary when nickel slag is used. However, using nickel slag, for both coarse aggregates can be an alternative way to improve material performance while cutting down on the use of natural aggregates [6].

Kadir et al. (2026) Studied how nickel slag can replace aggregate in concrete used to repair non-structural building parts. The study found that nickel slag met most of the needed tests and the concrete reached a compressive strength of 21.231 MPa after 28 days. This strength is below the standard for concrete but it is still good enough for non-structural parts. These results show that nickel slag can be an alternative aggregate for concrete helping to cut down on natural aggregate use and turning industrial waste into something helpful [7].

Fitriyanti and colleagues in 2025 studied the use of nickel slag as a replacement for aggregate in Self-Compacting Concrete. The study found that nickel slag, with No. 1 Gradation worked as fine aggregate. At the replacement level, it improved the compressive strength compared to the regular mix. It did not affect the self-compacting features. Using nickel slag can also help reduce the amount of sand needed. This approach supports the creation of environmentally friendly building materials [8].

Despite this growing work, few studies have looked at nickel slag from Southeast Sulawesi's ferronickel industry. Especially from PT VDNI. As a coarse aggregate substitute in Self-Compacting Concrete across all fresh-state Self-Compacting Concrete criteria and hardened-state physical and mechanical properties. Most existing substitution studies focus on vibrated concrete or a narrow set of performance indicators. This study fills that gap by giving an integrated evaluation of material characteristics, fresh Self-Compacting Concrete behavior and hardened concrete performance for a sourced nickel slag at three substitution levels.

Given the above the main question of this research is whether nickel slag can replace coarse aggregate (Split Moramo) in Self-Compacting Concrete. Partially or completely. While still meeting the EFNARC 2005 fresh-state acceptance rules and how such replacement changes the porosity, density and compressive strength of the hardened concrete. It is believed that even though flowability might drop because of the traits of nickel slag the mixtures will still stay inside acceptable Self-Compacting Concrete limits and that the harder denser nature of nickel slag will raise hardened-state mechanical performance compared with the natural-aggregate control.

The goal of this study is to evaluate the properties (slump flow, V-funnel and L-box) and hardened-state properties (porosity, density and compressive strength at 7 and 28 days) of Self-Compacting Concrete in which natural coarse aggregate is replaced with nickel slag from PT VDNI at replacement levels of 0%, 50% and 100%.

2. Materials and Methods

2.1. Materials

The cement used was Portland Composite Cement, with a specific gravity of 3.03 g/cm³, which is within the normal range of 3.0 to 3.2. The fine aggregate was river sand from Sirtu Pohara. Two kinds of aggregate were studied side by side: crushed stone from Split Moramo, which is the usual aggregate and nickel slag from PT VDNI, which is the substitute. A polycarboxylate-based superplasticizer was added to get the flow needed for Self-Compacting Concrete. The physical properties of the coarse aggregates were measured by standard tests and are shown in Table 1.

Table 1. Physical properties of the fine and coarse aggregates

Property	Moramo Split (Coarse Aggregate)	Nickel Slag (Coarse Aggregate)	Sirtu Pohara (Fine Aggregate)
Specific gravity, SSD	2.68	2.77	2.68
Water absorption (%)	0.61	1.15	0.61
Unit weight (Kg/L)	1.50	1.63	1.79
Los Angeles abrasion	35.50	23.00	-
Fineness modulus	6.88	6.82	2.55

As Table 1 shows, nickel slag has a specific gravity (2.77) and higher water absorption (1.15%) than Split Moramo (2.68 and 0.61%). This matches the fact that nickel slag's denser and has a more porous surface. The Los Angeles abrasion of nickel slag (23.00%) is much lower than that of

Split Moramo (35.50%) which means nickel slag is harder and more resistant to wear. The fineness modulus of both aggregates is close (6.82 for nickel slag versus 6.88 for Split Moramo), meaning both aggregates have a similar size distribution mainly around 9.5 mm.

2.2. Mix Design

The concrete mix design, for 1 m³ was decided by following the European Guidelines for Self-Compacting Concrete (EFNARC, 2005). Using the absolute volume method. The absolute volume method helped us find the proportion of each material by linking mass, specific gravity and absolute volume so that every part of the mixture could be counted in volume. The mixture has aggregate fine aggregate, cement, stone dust, water and superplasticizer. The specific gravities of aggregate fine aggregate and cement were 2.68, 2.55 and 3.03 as measured in this study. The specific gravities of stone dust and superplasticizer were 2.70 and 1.18 taken from Johan (2023) [4]. We used these gravity values to turn the needed mass of each material into absolute volume. Then we adjusted the mixture proportions to follow the EFNARC (2005) guidelines making sure the amounts of powder, aggregates, water and superplasticizer matched the needs of Self-Compacting Concrete (SCC). The calculations show that the total absolute volume of all materials is 1.01 m³. The final concrete mix design is shown in Table 2.

Table 2. Mixture Proportions of SCC for 1 m³ and EFNARC (2005) Specifications

Material	Weight	Specific Gravity	Volume (m ³)	EFNARC (2005) Specification
Coarse aggregate (Kg)	750	2.68	0.28	600-750 kg/m ³
Fine aggregate (Kg)	915	2.55	0.36	48-55% of total aggregate
Cement	420	3.03	0.14	380-600 kg/m ³
Stone dust (Kg)	150	2.70	0.06	
Water (Kg)	173	1.00	0.17	150-210 kg/m ³
Superplasticizer (L)	6.3	1.18	0.01	
Total			1.01	

Based on the mix design for 1 m³ the material quantities were scaled to produce eight cylindrical specimens (Ø10 × 20 cm 0.00157 m³ each) for each nickel slag substitution level. A 20% allowance was included to account for material losses during mixing, handling and casting. Therefore, the quantities presented in Table 2 represent the material requirements for one batch of eight specimens. The cement, stone dust, fine aggregate, water and superplasticizer contents were kept constant while the coarse aggregate was replaced by nickel slag at substitution levels of 0%, 50% and 100%. The resulting mixture proportions are presented in Table 3.

Table 3. Mixture proportions for the three substitution levels

Material	0% Nickel Slag (control)	50% Nickel Slag	100% Nickel Slag
Cement (Kg)	6.33	6.33	6.33
Stone dust Moramo (Kg)	2.26	2.26	2.26
Water (L)	2.61	2.61	2.61
Sirtu Pohara (Kg)	13.80	13.80	13.80
Moramo split (Kg)	11.31	5.65	0
Nickel slag (Kg)	0	5.65	11.31
Superplasticizer (L)	0.20	0.20	0.20

2.3. Specimen Preparation

Cylindrical specimens measuring Ø10 × 20 cm were cast for each mixture variation with three specimens prepared for each combination of test and curing age (7 and 28 days). Mixing followed a fixed sequence adapted to accommodate the properties of the substitute aggregate: the coarse aggregate (Split Moramo and/or nickel slag) was first combined with water and mixed until homogeneous; cement and Split Moramo stone dust were then added together with water; the fine aggregate (Sirtu Pohara) was subsequently introduced together with a portion of the superplasticizer pre-mixed with water. The remaining water and superplasticizer were added last to achieve the target flowability. In line with SCC practice specimens were cast without vibration followed by an initial curing period of 24 hours before demolding after which specimens were water-cured until the designated testing age.



Figure 1. Mixing of SCC materials using a concrete mixer

2.4. Testing Methods

Fresh-state properties of the SCC mixtures were evaluated in accordance with EFNARC 2005 guidelines comprising the slump flow test to assess filling ability the V-funnel test to assess viscosity and the L-box test (expressed as the blocking ratio $H2/H1$) to assess passing ability. State physical properties, Porosity and density. Were determined using a three-weighing method based on oven-dry weight saturated surface-dry (SSD) weight and submerged weight of the specimens at both 7 and 28 days. Compressive strength was determined by compression testing of cylindrical specimens at 7 and 28 days.

The slump flow test was conducted to evaluate the filling ability of the SCC mixture. The mixture was poured into a slump cone placed on a non-absorbent baseplate without any tamping or vibration. The cone was then lifted vertically in a smooth motion allowing the concrete to flow and spread freely under its own weight. The resulting spread diameter was measured in two directions and the average value was recorded as the slump flow value following the EFNARC (2005) procedure. The test setup is shown in Figure 2.



Figure 2. Slump flow test setup for evaluating the filling ability of fresh SCC



Figure 3. V-funnel test setup for evaluating the viscosity of fresh SCC

The L-box test was conducted to evaluate the ability of the fresh SCC mixture through congested reinforcement. The mixture was poured into the section of the L-shaped box while the sliding gate, fitted with vertical reinforcing bars remained closed. Once the vertical section was filled the gate was lifted to allow the mixture to flow into the section under its own weight. The height of the concrete remaining in the section ($H1$) and the height reached at the end of the horizontal section ($H2$) were then measured and the ratio $H2/H1$ was calculated as the passing ability index. The test setup is shown in Figure 4.



Figure 4. L-box test setup for evaluating the passing ability of fresh SCC

3. Result and Discussion

3.1. Characteristics of Materials

The material characterization results (Table 1) indicate that nickel slag possesses a harder particle structure than the natural Split Moramo aggregate, evidenced by its higher specific gravity and substantially lower abrasion loss. Its higher water absorption suggests a porous or irregular surface texture, which is consistent with the angular sub-vitreous morphology typical of granulated or air-cooled metallurgical slags. These characteristics are expected to influence both the fresh-state behavior of the SCC. Through increased friction and paste demand. And the hardened-state behavior, through improved interfacial bonding and mechanical interlock with the surrounding cement matrix.

3.2. Fresh Properties of SCC

The fresh-state properties of SCC at the three nickel slag substitution levels are presented in Table 4. The results show that all mixtures satisfied the EFNARC (2005) acceptance criteria for slump flow V-funnel time and L-box ratio.

Table 4. Fresh-state Properties of SCC at three nickel slag substitution level

Test EFNARC (2005)	0% Nickel Slag (control)	50% Nickel Slag	100% Nickel Slag	Specification
Slump flow (mm)	693.3	670.0	650.0	550-850
V-funnel time (s)	8	11	14	9-25
L-box ratio, H2/H1	0.85	0.82	0.80	0.8-1.0

A clear trend was observed with increasing nickel slag content. The slump flow diameter decreased progressively from 693.3 mm for the control mixture (0% nickel slag) to 670.0 mm at 50% substitution. 650.0 Mm at 100% substitution. In contrast the V-funnel time increased from 8 s for the control mixture to 11 s and 14 s at 50% and 100% substitution, respectively. These results indicate a reduction in flowability with increasing nickel slag content. This behavior can be related to the specific gravity and more angular particle shape of nickel slag, which can increase internal friction between aggregate particles and consequently reduce the flowability of the fresh SCC. The L-box ratio also decreased slightly from 0.85 for the control mixture to 0.82 and 0.80 at 50% and 100% nickel slag substitution respectively. The reduction indicates a decrease in passing ability as the nickel slag content increased. This behavior may be attributed to the angular shape of nickel slag particles, which can increase resistance as the concrete passes through the confined reinforcement. Nevertheless, the L-box ratios remained within the EFNARC (2005) recommended range of 0.80–1.00 indicating that all mixtures maintained passing ability.

3.3. Physical Characteristics of Hardened Concrete

The physical characteristics of hardened SCC were evaluated in terms of porosity and density at 28 days. The results for the different nickel slag substitution levels are presented in Table 5.

Table 5. Porosity and density of hardened SCC at different nickel slag substitution levels

Nickel Slag Substitution level	Porosity	Density
0%	12.09	2297.86
50%	11.77	2338.50
100%	11.38	2369.26

The density increased from 2297.86 kg/m³ for the control mixture (0% nickel slag) to 2338.50 kg/m³ and 2369.26 kg/m³ for the 50% and 100% nickel slag substitution levels, respectively. This increase is consistent with the specific gravity of nickel slag compared with Moramo split, resulting in a denser concrete matrix as the proportion of nickel slag increased. In contrast porosity decreased progressively with increasing nickel slag content from 12.09% for the control mixture to 11.77% at 50% substitution and 11.38% at 100% substitution. The reduction in porosity indicates a decrease in the volume of voids within the concrete. This trend can be related to the packing of nickel slag particles, which may reduce internal voids and contribute to the increase in density. The lower porosity observed at nickel slag substitution levels is also consistent with the increase in compressive strength indicating an overall improvement in the physical characteristics of the hardened SCC.

3.4. Compressive Strength

The compressive strength of SCC was evaluated at 7 and 28 days for three nickel slag substitution levels, as presented in Table 6.

Table 6. Compressive strength of SCC at three nickel slag substitution levels

Nickel Slag Substitution levels	7 days	28 days
0%	22.86	26.41
50%	24.93	28.05
100%	27.40	31.39

The results show a rise in compressive strength as nickel slag content grows for both testing ages. At seven days compressive strength rose from 22.86 MPa for the control mixture (0 % nickel slag) to 24.93 MPa and 27.40 MPa for the 50 % and 100 % nickel slag mixtures. At twenty-eight days compressive strength followed the pattern, rising from 26.41 MPa for the control mixture to 28.05 MPa for the 50 % nickel slag mixture and 31.39 MPa for the 100 % nickel slag mixture. Therefore, the 100 % nickel slag mixture reached the compressive strength at both testing ages. The 100 % nickel slag mixture achieved the compressive strength at both testing ages. The rise in strength with more nickel slag can be linked to the higher density and angular shape of nickel slag particles. These characteristics help create particle packing and stronger mechanical interlocking inside the concrete matrix. Moreover, the gradual drop in porosity when nickel slag content increases also supports the improvement in strength. The increase in strength with higher nickel slag content can be related to the higher density and angular particle characteristics of nickel slag, which contribute to denser particle packing and better mechanical interlocking within the concrete matrix. In addition, the progressive reduction in porosity observed with increasing nickel slag content supports the improvement in strength. Overall the

results show that using nickel slag as an aggregate can raise the compressive strength of SCC and the 100 % nickel slag mixture delivered the best strength among all mixtures studied.

4. Discussion

Taken together the results show a trade-off pattern. When nickel slag content grows, fresh-state flow parameters change a little – the slump flow spread becomes smaller the V-funnel time becomes longer and passing ability becomes slightly lower – but all of these changes stay well inside the EFNARC 2005 limits for all three substitution levels. At the time hardened-state performance gets better: density rises, compressive strength rises and porosity falls. These observations match the belief and the work of Candrika and Rahmad Zulfikar (2025) [2]. Destyanto and Wardhono (2018) [3] harder denser industrial by-product aggregates can be used in concrete without hurting workability while still giving measurable gains in mechanical performance. This study extends that idea to the case of SCC using nickel slag from PT VDNI in Southeast Sulawesi.

5. Conclusion

This study shows that nickel slag from PT VDNI can be a sound replacement for Split Moramo coarse aggregate in Self-Compacting Concrete even when the replacement level reaches 100 %. All tested mixtures met the EFNARC 2005 standards for slump flow, V-funnel and L-box even though flowability fell a little as nickel slag content grew. Hardened-state performance kept getting better as more nickel slag was used: density grew porosity fell and the 28-day compressive strength hit 31.39 MPa when 100 % of the aggregate was replaced – a rise of 18.8 % compared with the natural-aggregate control. These findings show that nickel slag is a sustainable and mechanically good alternative coarse aggregate for SCC giving a use for an industrial by-product while boosting concrete performance. More study is needed to look at the long-term durability of nickel-slag SCC, such as how it stands up to sulfate and chloride attack and to assess the cost aspects of using it on a large scale.

Nomenclature

SCC: Self-Compacting Concrete

SF : EFNARC 2005 classification for slump flow

VF : EFNARC 2005 classification for v-funnel

PA : EFNARC 2005 classification for passing ability l box

SSD: Saturated Surface Dry

Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Failure of Modular Steel Inter-Module Connections: A Review

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Abstract

Modular steel buildings rely on inter-module connections to transfer gravity, shear, and moment forces between volumetric units, and when these connections fail, the whole structure is at risk. This review examined 12 post-2008 studies and found that failure happens among a small set of recurring mechanisms: bolt slip or fracture, weld fracture, plate yielding or tearing, insufficient rotational stiffness, and connection distress during progressive collapse. Bolted and hybrid detachable systems are the most prevalent in the literature most studies on bolted, on hybrid or multi-system and 1 study on grouted –while no study focused exclusively on welded or post-tensioned systems, even though weld fracture appears repeatedly as a critical secondary limit state in bolted assemblies. This gap is important since the field has ignored the ones most prone to brittle failure while concentrating mostly on connection types that are easiest to assemble and test. The review sorts these findings into hierarchical limit-state categories: ductile plate yielding is the least harmful because it provides deformation capacity and visible warning, whereas brittle weld fracture and unseating-induced connection failures are the most critical because they occur suddenly and can eliminate the load path before the connection reaches full capacity. Beyond isolated strength, connection stiffness and deformation compatibility control how forces redistribute between adjacent modules, especially under lateral or accidental loading. Progressive-collapse studies confirm that connection performance must be assessed at the system level, considering alternative load paths, redundancy, residual resistance, and the location of module or column removal. All in all, the findings support designs that prevent premature fracture, maintain reliable force transfer, and concentrate damage in accessible, ductile, and replaceable components, but the lack of welded and post-tensioned studies are serious limitations that deter further generalization of design recommendations.

1. Introduction

Modular steel construction has expanded rapidly because it offers speed, quality control, and reduced on-site labor, but its structural reliability depends heavily on the joints between modules. Reviews of modular steel systems consistently note that inter-module connections remain one of the least mature parts of design practice, especially for lateral loading, cyclic loading, and robustness against accidental damage. Owing to the prefabricated nature of modular units, localized deficiencies within the connection interface can impair stress transfer, triggering damage propagation that compromises global frame stability. For that reason, failure of the connection is often treated as the key pathway to failure of the structure as a whole. This review draws mainly on two types of sources: review papers that classify inter-module connections, and experimental or numerical studies on collapse, stiffness, robustness, and joint behavior. Together, these sources sort inter-module connections into five categories: bolted, welded, grouted, self-locking, and hybrid. That categorization is useful, but it has a limitation. Two connections in the same category can fail in completely different ways depending on plate thickness, weld quality, bolt preload, and loading direction. Han et al. (2023) [5], for example, tested two bolted configurations and found that plate yielding governed the tensile tests while weld fracture governed the frame subassemblies. The connection type alone did not predict the failure mode (Corfar and Tsavdaridis 2022) [1].

To provide a more systematic basis for identifying research gaps, the cited literature was mapped quantitatively according to primary connection system: bolted, welded, grouted, and post-tensioned. Studies published between 2008 and 2025 were considered, consistent with the scope of this review. Each study was sorted by its main connection system. If a study covered more than one system, we placed it in a hybrid or emerging category. We then grouped the sources by what they mainly investigated bolt slip or fracture, weld fracture, plate yielding or fracture, stiffness degradation or slip, and progressive-collapse response. For instance, Han et al. (2023) [5] went into the bolted category because its

primary focus was a bolted connection, even though it also reported weld fracture. Table 1 summarizes the distribution of recent studies on inter-module connections. Of the 12 sources mapped, bolted systems represent the largest share (5 studies, 41.7%), followed by grouted systems (1 study, 8.3%). No study focused exclusively on welded or post-tensioned systems. Hybrid, multi-system, or general studies account for the remaining 6 sources (50.0%). The distribution reveals something important. Recent research has clustered around bolted and hybrid detachable connections, while welded and post-tensioned systems have received far less attention as primary research objects. Welded connections, in particular, appear mainly as a secondary concern—weld fracture shows up repeatedly as a critical limit state inside bolted or hybrid assemblies, but almost no study treats it as the main subject. This matters because it means connection classification by fastener type alone tells us very little. What actually governs failure is the interaction between the connection detail, the adjacent plates, the welds, and the loading scenario—not the fastener category on its own.

Why does this interaction matter so much? Because inter-module connections carry both vertical and horizontal actions between adjacent volumetric units. Their strength, stiffness, ductility, and deformation compatibility shape how demand spreads through the modular frame. Under cyclic lateral loading or local support-loss scenarios, this becomes critical. A connection that performs well under gravity load may still restrict force redistribution when an alternative load path is needed, which reduces the effectiveness of the remaining structure (Lacey et al. 2019 [2]; Alembagheri et al. 2020a [3]; Alembagheri et al. 2020b [4]).

Table 1. Quantitative distribution of recent studies on inter-module connections by primary connection system.

Primary connection system	Number of studies	Share (%)	Dominant performance/failure focus	Representative studies
Bolted	5	41.7	Bolt slip/fracture, plate yielding, weld fracture (secondary), rotational stiffness	Chen et al. (2019); Lacey et al. (2019); Han et al. (2023); Zhang et al. (2024); Yu et al. (2025)
Welded	0	0.0	Weld fracture appears only as a secondary limit state	Han et al. (2023); Yu et al. (2025)
Grouted	1	8.3	Sleeve bond, grout crushing, bar slip, partially rigid behavior	Dai et al. (2021)
Post-tensioned	0	0.0	Post-tensioning appears only in combination with bolted systems	Emamikoupaei et al. (2025)
Hybrid / multi-system / review / general	6	50.0	Self-locking, detachable joints, multi-system response, collapse robustness, connection classification	Alembagheri et al. (2020a); Alembagheri et al. (2020b); Corfar and Tsavdaridis (2022); Liu et al. (2024); Nadeem et al. (2021); Emamikoupaei et al. (2025)
Total	12	100		

Note: Each study is assigned to one primary system. Studies covering multiple systems (e.g., Emamikoupaei et al. 2025 [12], which includes bolted and post-tensioned connections) are listed under hybrid/multi-system. Reviews that focus on a single system (Lacey et al. 2019 [2]) are counted under that system; general reviews (Corfar and Tsavdaridis 2022 [1]; Nadeem et al. 2021 [11]) are counted under hybrid/general.

2. Categories of Connection Failures and Characterization

The reviewed literature is organized into five recurring connection-performance and failure categories: connection fracture and weld-related failure (Figure 1); bolt-related response and failure (Figure 3); plate yielding and plate fracture (Figures 4 and 5); stiffness degradation and slip (Figures 6 and 7); and progressive-collapse-related connection response (Figure 8). This classification distinguishes local component-level limit states from system-level robustness outcomes. In particular, progressive collapse is not a material failure mode equivalent to bolt fracture or plate yielding; it is a global structural response that may follow local connection damage or loss of load-transfer continuity (Alembagheri et al. 2020a [3]; Alembagheri et al. 2020b [4]; Corfar and Tsavdaridis 2022 [1]).

2.1. Fracture and Weld-Related Failure

Connection fracture and weld-related failure cover a range of problems: premature fracture of beam-to-column welds, fracture of connection plates, bolt-hole fracture, and sudden separation of the connection load path, as shown in Figure 1. These are critical failure modes because once local damage begins, the connection may lose its ability to develop further strength or deformation capacity (Han et al. 2023 [5]).

Han et al. (2023) [5] tested two bolted inter-module connection configurations using tensile pull-out tests and frame subassembly lateral cyclic tests (Figure 1). The tensile tests achieved plate yielding, which is the desired ductile response. But in the frame subassemblies, premature beam-to-column weld fracture controlled the ultimate behavior in every specimen. What is notable here is that a ductile inter-module connection detail did not translate into ductile system behavior. The adjacent welded beam-to-column joint remained the weak link. This is not an isolated finding—Yu et al. (2025) [6] reported similar concerns about weld behavior in their collapse experiments (Han et al. 2023 [5]; Yu et al. 2025 [6]).

So, this failure mode falls under connection fracture and weld-related failure under lateral frame loading. But the evidence does not support a general claim about which specific fracture type governs. Heat-affected-zone fracture, weld-metal fracture, and hollow-section wall fracture have all been suggested, yet no single mechanism dominates across all modular steel connections. What actually governs depends on the tested connection configuration, the loading arrangement, and where the fracture initiates. Han et al. (2023) [5], for example, observed beam-to-column weld fracture in their frame subassemblies—not heat-affected-zone cracking. That specificity matters. Designers should not assume one weld failure mode applies universally.

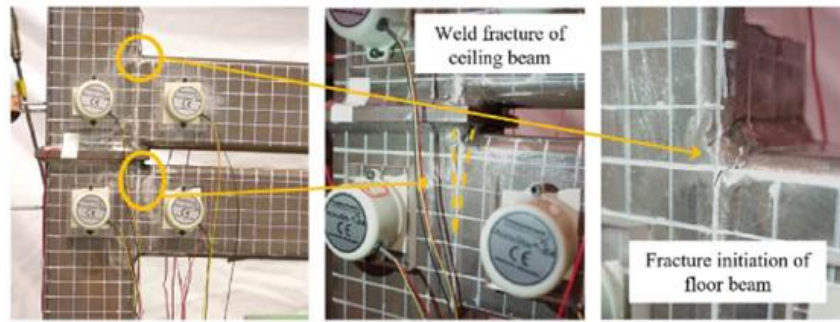


Figure 1. Weld Fracture Failure (Han et al. 2023) [5]

2.2. Bolt-Related Response and Failure

Bolt-related response includes bolt slip, bolt tension, bolt shear, bolt-hole bearing deformation, loss of clamping force, bolt fracture, and local deformation of connected plates around the bolt group. Bolted inter-module connections remain a major research focus because they allow rapid site assembly and can provide demountable or replaceable interfaces between modules (Corfar and Tsavdaridis 2022 [1]; Lacey et al. 2019 [2]). Lacey et al. (2019) [2] reviewed bolted inter-module connection systems and emphasized their importance in modular steel construction. In contrast, Han et al. (2023) [5] used component pull-out tests and frame subassembly tests to examine the tensile behavior, ductility, and failure sequence of specific bolted connection arrangements. Therefore, Lacey et al. (2019) [2] provide a broad classification of available bolted systems, whereas Han et al. (2023) [5] provide direct evidence of how the local connection response affects frame-level behavior. Under tensile loading, Han et al. (2023) [5] found that yielding of the end plate or anchoring plate was preferable to direct bolt failure because plate yielding provided greater deformation capacity. Liu et al. (2024) [7] also investigated the axial tensile response of a self-locking connection and showed that the connection behavior depended on the relative dimensions of the corner-fitting and tenon plate components. Unlike conventional bolted arrangements, the self-locking system was assessed primarily through axial tensile tests and numerical modelling rather than frame subassembly tests under lateral loading (Liu et al. 2024 [7]). Zhang et al. (2024) [8] examined a detachable tapered-head bolt connection under four-point bending and reported that the connection rotation resulted from deformation of the upper-corner bottom plate and elastic bolt deformation. In comparison with the tensile-focused study by Liu et al. (2024) [7], Zhang et al. (2024) [8] addressed flexural behavior and showed that the governing response of a detachable bolted connection may be controlled by plate bending rather than direct bolt fracture. These studies indicate that bolt behavior must be evaluated together with the stiffness and strength of the surrounding corner plates.

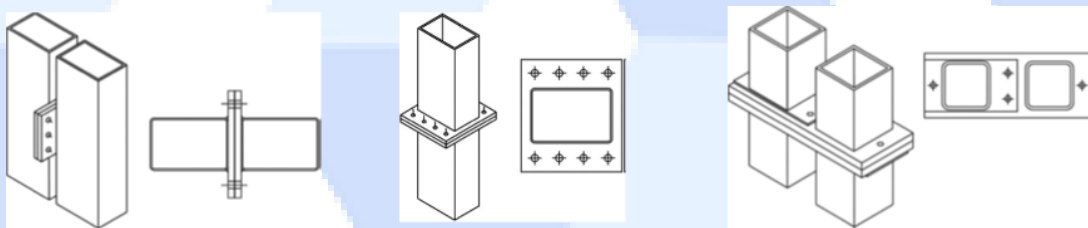


Figure 2. Bolt and plate arrangements (Lacey et al. 2019)



Figure 3. Bolt yielding and plate separation failure (Lacey et al. 2019)

2.3. Plate Yielding and Plate Fracture

Plate-related limit states include yielding, local buckling, tear-out, net-section fracture, and fracture around bolt holes in end plates (Figure 4), cover plates, corner plates, and anchoring plates. Stable plate yielding can be a favorable response when it occurs in an accessible and replaceable component and provides deformation capacity before a loss of connection resistance (Han et al. 2023 [5]; Zhang et al. 2024 [8]). Han et al. (2023) [5] found that plate yielding was preferable to bolt failure in tensile tests of their inter-module connections. Zhang et al. (2024) [8] similarly identified yielding of the upper-corner bottom plate as the principal failure mode of the detachable tapered-head bolt connection under bending. Both studies demonstrate that plate yielding serves as an effective ductile fuse under distinct loading protocols:

Han et al. (2023) [5] evaluated tensile and lateral cyclic responses (Figure 5), whereas Zhang et al. (2024) [8] investigated static flexural behavior under four-point bending. Dai et al. (2021) [9] investigated grouted sleeve connections under bending and compared shear-keyed and slot-hole connection configurations. Both connection types provided moment resistance and ductility sufficient for partially rigid behavior in braced modular steel structures (Dai et al. 2021 [9]). In contrast with the dry detachable connection studied by Zhang et al. (2024) [8], the grouted sleeve system transfers forces through a grouted interface rather than through a detachable tapered-head bolt arrangement. This difference is important because the connection force-transfer mechanism, construction process, damage location, and repair strategy differ substantially between grouted and dry mechanical systems. Plate yielding should not automatically be described as harmless or fully desirable. Stable yielding is beneficial only when it does not progress into local plate buckling, tear-out, net-section fracture, or connection separation. In the collapse experiments by Yu et al. (2025) [6], bolt-hole fracture occurred in the side-column-loss specimen but was not reported for the corner-column-loss specimen, demonstrating that the same connection detail can develop different local damage mechanisms under different support-loss scenarios (Yu et al. 2025 [6]).



Figure 4. Vertical plate failure (Zhang et al. 2024 [8])

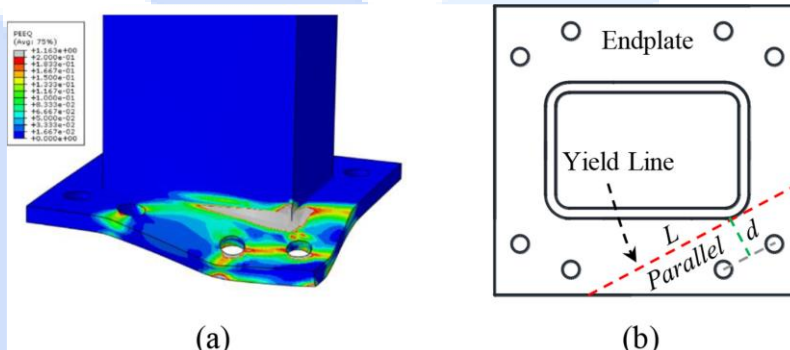


Figure 5 Horizontal plate yielding (Han et al. 2023 [5])

2.4. Stiffness Degradation and Slip

Not all unsatisfactory connection responses involve complete material fracture. Excessive slip, insufficient rotational stiffness, loss of interface stiffness, and poor deformation compatibility can impair the lateral and gravity-load response of a modular steel building even when the connection has not reached its ultimate strength (Chen et al. 2019 [10]; Corfar and Tsavdaridis 2022 [1]). Chen et al. (2019) [10] investigated the initial and post-yield rotational stiffness of inter-module joints in mid-rise modular steel buildings. Their work focused on the role of connection rotational stiffness in the overall building response. By comparison, Dai et al. (2021) [9] used bending tests to demonstrate that grouted sleeve connections could provide partially rigid behavior (Figure 6). Chen et al. (2019) [10] therefore addressed rotational stiffness as a system-analysis parameter, while Dai et al. (2021) [9] evaluated the flexural response and force-transfer mechanism of a specific connection detail. A connection may retain vertical load-carrying capacity while still providing inadequate lateral or rotational restraint. In such cases, excessive joint deformation can increase story drift, modify the assumed load path between modules, amplify second-order effects, and increase the demand on adjacent members and connections (Chen et al. 2019 [10]; Nadeem et al. 2021 [11]). Stiffness degradation and slip should consequently be treated as serviceability and structural-performance concerns, rather than only as precursors to ultimate connection failure.

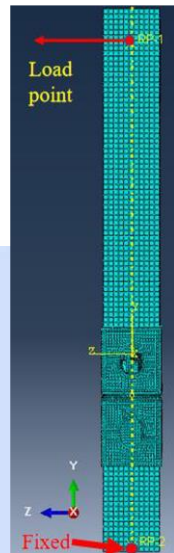


Figure 6. Rotational stiffness failure (Chen et al. 2019) [10]



Figure 7. Rotational stiffness testing (Chen et al. 2019) [10]

2.5. Progressive-collapse-related connection response

Progressive-collapse-related connection response concerns the ability of inter-module connections to sustain alternative load paths after local damage, including module loss, corner-column removal, or side-column removal. The relevant performance measures include residual load capacity, connection force demand, deformation capacity, relative module displacement, and the ability to redistribute loads around the damaged region (Alembagheri et al. 2020a [3]; Alembagheri et al. 2020b [4]; Yu et al. 2025 [6]). Alembagheri et al. (2020b) [4] investigated module-loss scenarios using nonlinear analysis with horizontal and vertical inter-module connections represented by nonlinear axial and shear springs. Their study showed that the connection properties strongly influenced collapse capacity and load redistribution after module loss. Alembagheri et al. (2020a) [3] further reported that the translational response of inter-module connections, particularly their shear behavior, had a greater influence on collapse resistance than rotational stiffness when the individual modules behaved as rigid moment frames. Yu et al. (2025) [6] adopted a different approach by conducting comparative collapse experiments on modular substructures subjected to corner-column loss and side-column loss. The tests showed different deformation patterns and failure mechanisms between the two removal scenarios, including bolt-hole fracture in the side-column-loss specimen and substantial local buckling at beam ends away from the removal region. Thus, Alembagheri et al. (2020a; 2020b) [3,4] provide system-level numerical evidence of alternative load-path development, whereas Yu et al. (2025) [6] provide experimental evidence that the local failure mechanism depends strongly on the location of the removed support. Emamikoupaei et al. (2025) [12] extended this system-level perspective through nonlinear static pushdown and fragility analyses of five-, ten-, and fifteen-story modular steel buildings with bolted and post-tensioned-rod inter-module connections. Under module-removal scenarios, their findings indicated that taller buildings exhibited superior robustness due to higher structural indeterminacy and alternate load paths. Unlike the component and substructure studies, this work evaluates the probability of progressive collapse as a function of connection type and building height (Emamikoupaei et al. 2025 [12]).

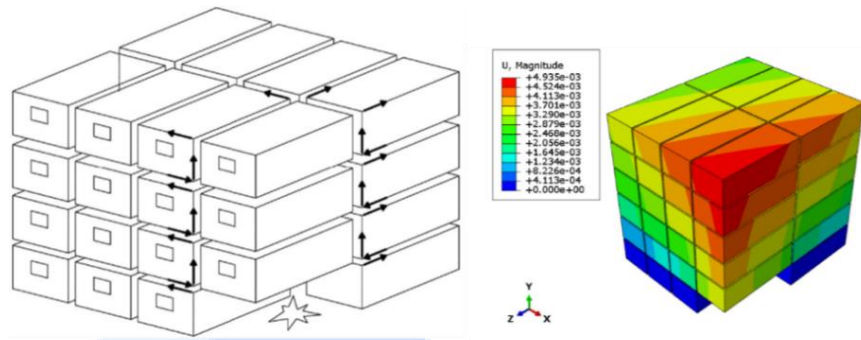


Figure 8. Load redistribution in modular buildings due to module removal (Alembagheri et al. 2020b [4])

3. Critical Discussion

The reviewed studies indicate that bolted inter-module connections remain the most common failure mechanism in modular steel construction, largely because of their practical assembly advantages and their widespread use in both conventional and emerging modular connection systems (Corfar and Tsavdaridis 2022 [1]; Lacey et al. 2019 [2]; Nadeem et al. 2021 [11]). This research concentration should not be interpreted to mean that bolted connections are inherently weaker than welded, grouted, or self-locking systems. Instead, it reflects the maturity of the available connection typologies and the number of experimental and numerical studies focused on their force-transfer mechanisms. The principal difference between the studies is the scale and loading condition of the investigation. Han et al. (2023) [5] combined connection-level tensile tests with frame subassembly tests, thereby linking local plate yielding and bolt response to frame-level weld fracture. Zhang et al. (2024) [8] focused on static bending of a detachable bolted connection and identified ductile yielding of the upper-corner bottom plate. Dai et al. (2021) [9] examined grouted sleeve connections under bending and showed partially rigid behavior without relying on conventional site bolting or welding. These studies cannot be compared solely by ultimate strength because their connection geometries, load paths, assembly methods, and loading protocols are different. System-level robustness assessments confirm that localized connection capacity alone cannot adequately characterize progressive collapse resistance. Alembagheri et al. (2020a; 2020b) [3,4] showed that inter-module connection characteristics influence alternative load-path development after module loss, while Yu et al. (2025) [6] demonstrated experimentally that corner- and side-column loss can produce distinct deformation patterns and local damage mechanisms. Emamikoupaie et al. (2025) [12] further showed that building height, redundancy, and connection type influence fragility-based robustness under progressive-collapse scenarios. These comparative trends are synthesized in Table 2, which summarizes the failure modes, initial rotational stiffness, ductility ratios, and demountability of the principal inter-module connection types, highlighting the trade-offs between stiffness, ductility, and reusability that should guide connection selection in modular steel construction.

Table 2. Comparative synthesis of inter-module connection (IMC) types by failure mode, initial rotational stiffness, ductility ratio, and demountability

Connection type	Failure mode	Initial rotational stiffness	Ductility ratio	Demount ability
Bolted	Bolt slip, bolt fracture, plate yielding/tearing	Moderate to high (semi-rigid; depends on bolt grade, preload, and plate thickness)	Moderate to high (ductile when plate yielding or bolt deformation governs; low when bolt shear fracture governs)	High (fully demountable; bolts can be loosened and components reused)
Welded	Weld fracture, heat-affected-zone cracking, plate tearing	High (approaching rigid; governed by weld geometry and continuity plates)	Low to moderate (brittle weld fracture limits ductility; ductility improves if plate yielding precedes weld failure)	Low (generally non-demountable; requires cutting and re-welding)
Grouted sleeve	Grout crushing, sleeve rupture, bar pull-out/slip	Moderate (semi-rigid; governed by sleeve length and grout strength)	Low to moderate (bond-slip and grout crushing may reduce ductility; adequate confinement improves performance)	Low (non-demountable after grouting; sleeve and bar embedded)
Post-tensioned	Tendon yielding, anchorage failure, strand slip, unseating	Low to moderate (semi-rigid; stiffness recoverable via re-tensioning)	High (self-centering behavior provides large deformation capacity with limited residual drift)	Moderate to high (partially demountable; tendons can be de-stressed and reused, but anchorage components may need replacement)
Self-locking / improved detachable joints	Locking mechanism disengagement, local bearing failure, plate yielding	Low to moderate (variable; depends on interlocking geometry and contact bearing)	Moderate (ductility governed by locking component deformation; premature disengagement reduces ductility)	High (designed for rapid assembly and disassembly; reusable with inspection)

4. Conclusion

The reviewed studies show that the critical connection concerns in modular steel construction are not limited to a single failure mode. The most consequential behaviors are premature weld fracture, direct bolt failure, plate fracture or bolt-hole fracture, excessive slip, insufficient rotational stiffness, and loss of force-transfer continuity after local damage.

Stable yielding of an end plate, anchoring plate, or corner plate is generally more favorable than abrupt bolt or weld fracture because it can provide deformation capacity and visible warning before complete loss of resistance. However, plate yielding is beneficial only if the yielding remains stable and does not progress into tear-out, local buckling, net-section fracture, or connection separation.

For prefabricated modular steel buildings, the connection design objective should be to maintain reliable axial, shear, and moment transfer; prevent premature fracture; provide adequate rotational and translational stiffness; and promote controlled, repairable damage in ductile and accessible components. Under accidental module-loss or column-loss scenarios, the inter-module connections must also sustain the deformation demands needed to activate alternative load paths and maintain overall structural robustness.

Declaration of Conflict of Interests

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

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Effect of Fine Recycled Concrete Aggregate on the Engineering Properties of Low-Molarity Fly Ash-GGBS Geopolymer Mortar

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Keywords

Abstract

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concrete aggregate,
Type M mortar.*

The high carbon dioxide (CO₂) emissions from Ordinary Portland Cement (OPC) production and the rapid accumulation of construction waste demand sustainable alternatives. This study examines the engineering behaviour of a geopolymer mortar in which OPC is replaced entirely by a blended aluminosilicate precursor (80% fly ash-20% ground granulated blast-furnace slag, GGBS) and part of the natural sand by treated fine recycled concrete aggregate (TFRCA). Six mixes were prepared at 8M and 10M sodium hydroxide, with the liquid-to-binder ratio held at 0.43, TFRCA at 0%, 10% and 25% by weight, and curing at ambient temperature. Flow, compressive strength at 1, 3, 7 and 28 days, and porosity were measured. The 28-day strength ranged from 27.82 MPa to 37.45 MPa, so every mix satisfied the 17.2 MPa required of a Type M mortar in ASTM C270 by a margin of 62% to 118%, and exceeded class CS IV of BS EN 998-1 by more than four times. Raising the molarity returned only 2.06 MPa at 10% TFRCA but 7 MPa at 25%. Porosity rose from 11.7% to 15.6% at 8M and from 11.3% to 13.5% at 10M as the recycled content increased, while flow fell by as much as 28%. An 8M activator is therefore sufficient at every replacement level tested; 10M is justified only where strength above the class requirement is specified. A low-molarity, ambient-cured fly ash-GGBS mortar carrying up to 25% treated recycled fines is a workable alternative for general construction.

1. Introduction

Ordinary Portland Cement (OPC) is the predominant construction binder; however, its manufacture contributes approximately 8% of global anthropogenic carbon dioxide (CO₂) emissions, largely because of the high-temperature calcination of limestone and the associated fuel combustion [1]. At the same time, rapid urbanization has accelerated the depletion of natural river sand and the accumulation of construction and demolition (C&D) waste, much of which is still disposed of in landfills rather than being reused [2]. These two pressures, carbon-intensive binder production and growing C&D waste, have created an urgent need for alternative binding materials and aggregate sources that can deliver comparable mechanical performance with a substantially lower environmental footprint.

Geopolymer technology, first conceptualized by Davidovits, offers a promising alternative binding mechanism. Instead of relying on OPC hydration chemistry, an aluminosilicate-rich industrial by-product is activated by a concentrated alkaline solution, typically sodium or potassium hydroxide combined with a silicate, to trigger a geopolymerization reaction. The reaction proceeds through dissolution of the binder, followed by reorientation and polycondensation of the dissolved silica and alumina species into a rigid, three-dimensional sodium aluminosilicate hydrate (N-A-S-H) gel network that gives the hardened material its strength [3].

Fly ash, a by-product of coal combustion in power plants, is the most extensively studied geopolymer binder because of its high amorphous silica and alumina content, spherical particle morphology and wide availability. Hardjito et al. [4] showed that fly ash-based geopolymer concrete can achieve compressive strengths comparable with OPC concrete, but also noted that low-calcium (Class F) fly ash reacts relatively slowly at room temperature and commonly requires elevated-temperature curing to reach its full strength potential within a practical timeframe. Because heat curing is impractical for site-based work, researchers have blended fly ash with more reactive secondary precursors, such as ground granulated blast-furnace slag (GGBS), to enable ambient-temperature curing. The calcium supplied by GGBS promotes the parallel formation of a calcium-silicate-hydrate (C-S-H) gel alongside the N-A-S-H geopolymer gel, which densifies the microstructure and accelerates early strength development [5]. Comprehensive reviews of geopolymer mortar have similarly highlighted that binder combination and activator concentration are the dominant factors controlling its mechanical suitability for field application [6].

In parallel with binder development, the gradual depletion of aggregate has pushed the construction industry toward alternative aggregate such as the adoption of recycled concrete aggregate. Guo et al. [7] reported that untreated recycled concrete aggregate is inherently more porous than natural sand because of the old, adhered mortar remaining on its surface, which increases water absorption, weakens the interfacial

transition zone, and consequently lowers the mechanical strength and durability of the resulting mortar or concrete. Surface treatment methods, such as densification with a sodium silicate solution or other pozzolanic coatings, have been shown to reduce this residual porosity and improve the compatibility of recycled fines with the surrounding binder matrix [8]. Within alkali-activated systems specifically, Liu et al. [9] demonstrated that the quality of the recycled fine aggregate has a measurable influence on the strength development of fly ash/GGBS-based geopolymer mortar, reinforcing the need to evaluate treated recycled aggregate under low-molarity, ambient-cured conditions rather than relying solely on findings from OPC-based systems.

From an application perspective, geopolymer mortar is increasingly considered for alternative cement-based works because of its comparable strength capability to cement-based composites. CS IV, the highest compressive strength class of BS EN 998-1 for rendering and plastering mortar, requires a minimum of 6 N/mm² at 28 days [10] which is acceptable. ASTM C270 is similar for unit masonry, reserving the highest of its four types, Type M, for a mortar that achieves a minimum 17.2 MPa at 28 days [11]. A binder that achieves the same without any Portland cement and would also use recycled fines would be directly applicable to load-bearing masonry and for rendering applications. The fly ash/slag geopolymer composites are also known to be superior in bonding capacity with concrete substrates, benefitting when the mortar is applied over the existing element [12]. The study by Nadarajah et al. [13] on ambient cured fly ash-GGBS geopolymer mortar shows that the proportioning of the binder and the concentration of the activator determine the strength level of the geopolymer mortar as required in existing mortar standards.

Despite this growing body of work, the combined effect of using treated fine recycled concrete aggregate (TFRCA) together with a low-molarity (8M-10M) NaOH-activated, ambient-cured fly ash-GGBS binder system on the mechanical performance of geopolymer mortar remains insufficiently quantified. Addressing this gap, the present paper reports the mechanical performance of a fly ash-GGBS geopolymer mortar in which OPC is completely replaced by an 80% fly ash-20% GGBS blend activated with low-molarity NaOH (8M and 10M), and natural sand is partially replaced with TFRCA at 10% and 25%. This study determines how activator molarity and TFRCA replacement levels affect flowability, compressive strength development and porosity of the mortar, and establishes the strength class that the material can be assigned for use as a sustainable alternative in construction applications.

2. Testing process

2.1. Materials

Low-calcium (Class F) fly ash was used as the primary aluminosilicate binder, supplied by a local slag-cement producer. Ground granulated blast-furnace slag was used as the secondary binder, blended with fly ash at a fixed ratio of 80% fly ash to 20% GGBS by binder weight. The activator combined sodium silicate with sodium hydroxide solution at a sodium silicate to sodium hydroxide ratio of 2.5 and was prepared at least 24 hours before casting at molarities of 8M and 10M. Same dosage of the naphthalene based superplasticiser was used (3% by binder weight) to assist workability. Natural river sand and treated fine recycled concrete aggregate (TFRCA) were used as fine aggregate, both sieved to a 1.18 mm-300 μ m particle size range. The recycled aggregate was treated by soaking in a 5% sodium silicate solution for 1 hour, oven-dried for 24 hours, and air-cured for 1 day [8].

2.2. Mix Design

The study was conducted on a single fly ash-GGBS binder system with different amounts of TFRCA (10% and 25% by weight) and 8M or 10M of NaOH solution for the activation of the geopolymer. The liquid-to-binder ratio was kept constant at 0.43, and the sodium silicate-to-sodium hydroxide (SS/SH) ratio was fixed at 2.5. The mix variables are summarised in Table 1 and the six mixes are listed in Table 2. The specimens were cast in oiled moulds, vibrated to extrude air, demoulded after 24 hours and left to cure until testing for compressive strength and porosity at 1, 3, 7 and 28 days at ambient temperature.

Table 1. Geopolymer mortar parameters

Binder System	Binder Ratio	NaOH Molarity	TFRCA Replacement
Binary blend	80% FA – 20% GGBS	8M, 10M	0%, 10%, 25%

Table 2. Mix combinations of geopolymer mortar

No	Mix
A1	80%FA-20%GGBS (100%NA, 8M)
A2	80%FA-20%GGBS (10%TFRCA, 8M)
A3	80%FA-20%GGBS (25%TFRCA, 8M)
B1	80%FA-20%GGBS (100%NA, 10M)
B2	80%FA-20%GGBS (10%TFRCA, 10M)
B3	80%FA-20%GGBS (25%TFRCA, 10M)

2.3. Mechanical Testing

The flowability was determined on fresh mortar after mixing according to the principle of flow table test as per ASTM C1437 and BS EN 1015-3 [14,15]. Compressive strength was determined in accordance with BS EN 12390-3 (2019) [16] using 50 mm mortar cubes tested with a compression machine. Two cube specimens were tested for each mix at the ages of 1, 3, 7 and 28 days. The porosity was determined from 28-day cubes using the water displacement method of ASTM C642 [17].

3. Results and Discussion

3.1. Flowability

The flow of each of the six geopolymer mixes is presented in Fig. 1. With 100% natural sand, the 8M mix spread to 153.0 mm, a flow of 118.6%, while the same mix activated at 10M spread to 146.8 mm, or 109.6%. This is due to the increase in viscosity of the more concentrated hydroxide solution since more of the water molecules are associated with sodium ions and cannot be used for lubrication of the solid particles [18].

In all cases, TFRCA lowered the flowability. At 8M the flow fell from 118.6% to 86.4% at 10% replacement and to 85.4% at 25%, and at 10M from 109.6% to 96.8% and then 90.4%. There are two factors at work here. The treated recycled fines absorb 5.42% water, which is more than the river sand with 1.64%, so some of the activator solution is absorbed by the aggregate and the rougher surface of the treated recycled fines is angular, which increases the internal friction between particles [7,9]. The sodium silicate treatment does not eliminate the absorption properties of the old mortar that is to be adhered, but rather hardens the surface of the old mortar [8].

All tested mortars were cohesive, allowing for manual placement and compaction without segregation or bleeding, which is consistent with reports that blending GGBS into a fly ash system improves the cohesiveness of ambient-cured mixes [5] and shows that the loss of workability is not a serious problem when using such mortars. It is also noted that the highest flow is the 8M mix using natural sand, which is beneficial when the mortar needs to be spread and finished as a plaster or a render.

3.2. Compressive Strength

Fig. 2 presents the compressive strength development of all six-fly ash-GGBS mixes at 1, 3, 7 and 28 days. Among the 100% natural sand mixes, the blend activated at 8M NaOH (A1) reached 30.88 MPa at 28 days, reflecting the calcium supplied by GGBS promoting early co-precipitation of a calcium-silicate-hydrate gel that densifies the matrix alongside the N-A-S-H geopolymer gel [5].

Increasing the activator concentration from 8M to 10M substantially improved strength in 100% natural sand, the compressive strength rose from 30.88 MPa (A1, 8M) to 37.45 MPa (B1, 10M), the highest 28-day compressive strength. The additional hydroxyl ions available at 10M accelerate dissolution of the reactive calcium, silica and alumina species, promoting a more complete and denser polycondensation reaction, a trend also reported for slag-blended geopolymers activated at increasing NaOH concentration [19].

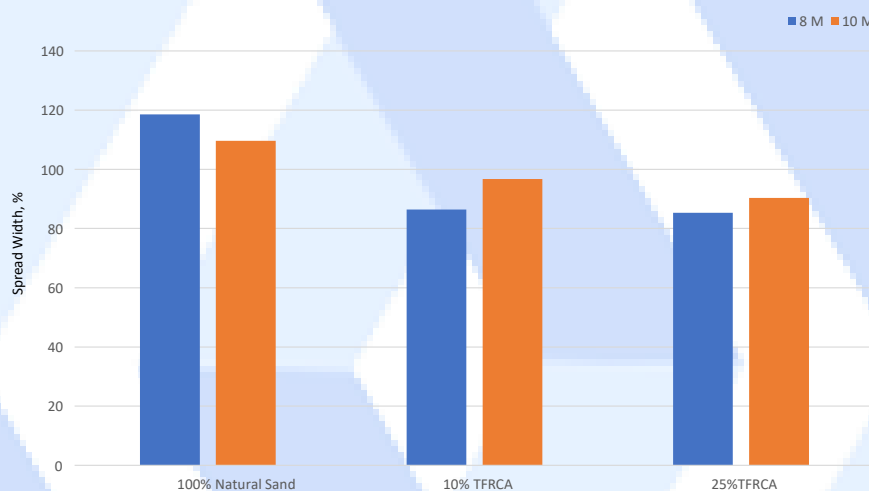


Figure 1. Flow of the geopolymer mortar at 8M and 10M for 0%, 10% and 25% TFRCA

When natural sand was replaced with TFRCA, the compressive strength of the fly ash-GGBS mixes decreased. With 10% TFRCA, the mix activated with 8M and 10M NaOH reached 31.00 MPa (A2) and 33.10 MPa (B2) after 28 days. At 25% TFRCA, the strength dropped further, but the mix with 10M NaOH still reached 34.80 MPa (B3). This suggests that a denser geopolymer gel network at higher molarity can partly compensate for the weaker structure caused by the more porous recycled aggregate, similar to what has been seen in recycled-aggregate fly ash/GGBS geopolymer mortar [9].

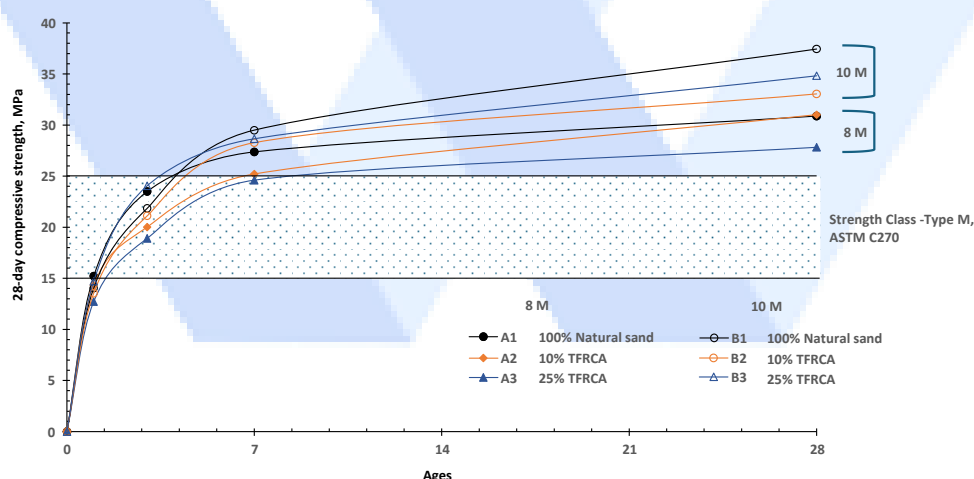


Figure 2. Compressive strength development of the six mixes

The strength region for Type M mortar, the highest type covered by ASTM C270, which requires 17.2 MPa at 28 days [11], is also shown in Fig. 2. All mixes performed above that range (in the case of A3, it is 62% higher, and in the case of B1, it is 118% higher). The blend can therefore be classified as a high strength mortar produced without Portland cement and without heat curing, and the classification still holds when 25% of the natural sand is replaced by TFRCA.

Comparing the mixes in pairs shows what the extra alkali buys. At 10% TFRCA the step from A2 to B2 is 2.06 MPa, or 6.6%, no more than the scatter of a two-specimen average; at 25% it is 7.00 MPa, or 25.2%. A1, A2 and A3 reached 30.88, 31.00 and 27.82 MPa, clearing the Type M requirement by 80%, 80% and 62%. Therefore, an 8M is adequate for all replacement levels investigated and a 10M should only be used if a job requires more than the class requires. Working at 8M also means 20% less sodium hydroxide to purchase, store and handle. These mixes are suitable for plaster and render work. According to BS EN 998-1 which classifies the rendering/ plastering mortar, CS IV is the highest, requiring only 6 N/mm² [10]. Strength is never the controlling factor in a plastering application, since A3, the weakest mix here, exceeds that limit by more than four times. Consistency is what governs: the 8M mixes containing recycled fines flowed at 85.4% and 86.4%, against 90.4% and 96.8% at 10M, so the superplasticiser dosage should be raised slightly or the fines pre-saturated before the mortar is spread on a wall.

3.3. Porosity

The volume of permeable voids as measured at 28 days is shown in Fig. 3. Raising the activator concentration lowered porosity at every replacement level of TFRCA, from 11.67% to 11.29% at 0% TFRCA, from 13.33% to 12.00% at 10% TFRCA, and from 15.57% to 13.45% at 25% TFRCA, reductions of 3.3%, 10.0% and 13.6% respectively. A higher concentration hydroxide solution will dissolve more of the aluminosilicate precursor, thus producing more N-A-S-H and C-S-H gel to fill in the void spaces between the aggregate particles and leaving a finer and less connected capillary network.

The porosity increased by 14.2% and 33.4% for 10% and 25% replacement of TFRCA at 8M and 6.3% and 19.1% at 10M replacement respectively for both combinations. The old mortar which sticks to the recycled fines is already porous, and the sodium silicate treatment does not remove voids within the old mortar. The much smaller growth in the 10M combinations is important as it indicates that the additional gel that is produced at the higher molarity not only accommodates the added porosity created by the recycled aggregate, but some of it also diminishes the porosity. Despite the increment in porosity, the strength achievement is within the range of the Type M requirement. Nonetheless, the porosity obtained indicates a good indicator for durability.

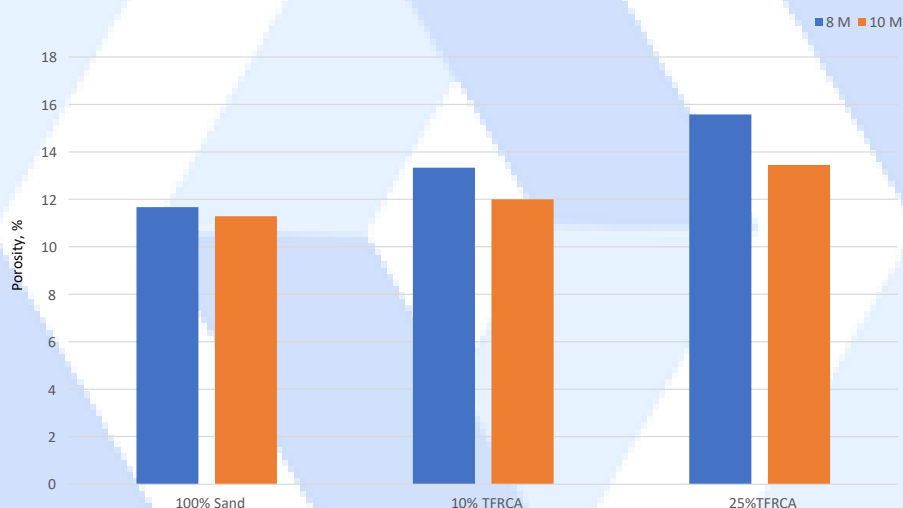


Figure 3. Porosity of the geopolymer mortar at 28 days for 0%, 10% and 25% TFRCA

4. Conclusion

From the study, the following conclusions can be drawn:

- The 28-day strength of all six 80%FA-20%GGBS mortars at ambient temperature was between 27.82 and 37.45 MPa, which are above the 17.2 MPa specified by ASTM C270 for a Type M mortar.
- Mixes are suitable for plaster and render. It is consistency, not strength that dictates a plastering application and Class CS IV (highest rendering and plastering) requires just 6 N/mm².
- Replacing natural sand with TFRCA increased porosity in every case, by 14.2% and 33.4% at 8M and by 6.3% and 19.1% at 10M for 10% and 25% replacement, and reduced the flow by as much as 28%. The 28-day strength was affected less than the porosity, so a 25% replacement level remains acceptable on the basis of strength achievement.

Nomenclature

OPC: Ordinary Portland cement

FA: Fly ash

GGBS: Ground granulated blast-furnace slag

NA: Natural aggregate (natural river sand)

TFRCA: Treated fine recycled concrete aggregate

C&D: Construction and demolition

NaOH: Sodium hydroxide

M: Molarity of the sodium hydroxide solution (mol/L)

SS/SH: Sodium silicate to sodium hydroxide ratio

L/B: Liquid-to-binder ratio

N-A-S-H: Sodium aluminosilicate hydrate gel

C-S-H: Calcium silicate hydrate gel

CS IV: Highest compressive strength class of BS EN 998-1 for rendering and plastering mortar

Type M: Highest mortar type of ASTM C270 for unit masonry (minimum 17.2 MPa at 28 days)

Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Optimization of AC-BC Hot Mix Asphalt Incorporating Granular Asbuton and Epoxy Resin

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*AC-BC hot mix asphalt,
Granular Asbuton,
Epoxy resin,
Marshall characteristics,
Indirect tensile strength.*

Abstract

Epoxy resin has increasingly been used as a binder modifier in asphalt mixes because epoxy resin may boost road performance and improve the physical characteristics of asphalt materials. This study aims to check how adding epoxy resin changes the properties of 60/70 penetration-grade asphalt Marshall characteristics and indirect tensile strength (ITS) of Asphalt Concrete-Binder Course (AC-BC) mixtures. A laboratory experiment was conducted by adding epoxy resin as a substitute for usual oil-based asphalt at several set amounts. First, the asphalt binder properties were checked to see the results of epoxy resin modification; then Marshall testing was done to evaluate how stable the mix is, how it flows, how much air it holds, and how well it works overall. Also, the indirect tensile strength test was carried out to check how strong the AC-BC mixtures are when pulled. The experimental results show that adding epoxy resin can make AC-BC mixtures that meet the requirements of the 2025 General Specifications of Bina Marga. The changed 60/70 penetration-grade asphalt and the resulting Marshall characteristics were also found to stay within the specification limits. The best ITS value occurred when epoxy resin was 5%, indicating that the pull strength improved compared with mixes that had lower epoxy resin amounts. However, looking at the balance of binder and mix performance, the best epoxy resin amount was found to be 2.4% together with the best asphalt amount (OAC) of 6.1%. These findings show that replacing asphalt with epoxy resin can improve the performance of AC-BC mixtures while complying with the applicable road material specifications.

1. Introduction

Road pavement is an important part of transportation systems. It helps people move around and get things from one place to another. It helps communities grow. Because of this, the quality of the road pavement must be thought about carefully. This is so that people can travel safely, feel comfortable, and the roads can last a long time. One way to make roads better is to use materials and things added to the asphalt. These can make the asphalt stronger and more stable. Choosing the materials is also important. It should take into account whether the materials are easy to get, how well they work, if they are good for the environment, and if they can use things that're already nearby. This helps make roads that work well, cost less, and can handle all kinds of traffic and weather [1]. In line with the development of materials and the use of local resources, Asbuton or Buton natural asphalt is seen as a possible material for building roads. Asbuton is found naturally on Buton Island, which's in Southeast Sulawesi. There are amounts of Asbuton estimated at about 677.247 million tons. This number comes from data collected by the Southeast Sulawesi Provincial Mining and Energy Office in 1997. This big amount of Asbuton offers a chance to use Asbuton as a material. Using Asbuton can help create sustainable road systems [2].



Figure 1. Granular Asbuton

Because there is plenty of Asbuton I think it looks very promising as a material that can help with long-term road construction and maintenance. The huge natural asphalt reserves in Southeast Sulawesi give us a chance to cut down on the use of petroleum-based asphalt. Using Asbuton still has some problems, mainly because Asbuton has a low bitumen content and low bitumen penetration. These traits can hurt how asphalt mixes behave, their flexibility, and how long the pavement lasts. So we need development and creative solutions to improve the characteristics of Asbuton and make it work better as a road pavement material [3].

To make the use of Asbuton and fix the problems with its bitumen, it is important to use special materials that can make the asphalt mix better. One possible change is to use a polymer, ** epoxy resin **, which can work as a binder and make the mix stronger and more durable when it is used on roads and exposed to weather. In this research, epoxy resin is added to the asphalt mix to make the Asbuton-based road better so that this material which is easy to get can be used in the best possible.

Several past studies have looked at using Asbuton and epoxy resin as materials or as additions in asphalt pavement mixtures. Ardhian et al. Studied Asbuton from Lawele as a partial replacement for Pen 60 asphalt in Laston mixtures. They used ratios from 65%–35% to 80%–20%. The results showed that the mixtures with Asbuton met the needed stability requirements. The best amount of asphalt used was 6% [4].

Cong et al. Examined epoxy resin as a modifier for asphalt in order to boost the performance and mechanical properties of the binder. The results showed that epoxy asphalt initially had compatibility and low flexibility. However, when a solubilizer and a styrene-butadiene-styrene (SBS) polymer was added, epoxy asphalt gained compatibility, its morphology improved, and its elongation increased. These findings highlight how polymer-based materials can raise the performance of epoxy asphalt for use in roads [5].

Salsabil and colleagues looked at adding epoxy resin and Dramix steel fibers to AC-WC mixtures that use PG-70 rubber asphalt. They found that using 4 percent epoxy resin with 4 percent Dramix steel fibers gave the best VIM, VMA, and Marshall stability numbers: 4.92 percent, 16.79 percent and 1,357.13 kilograms. I think this shows that adding epoxy resin and Dramix steel fibers can improve the Marshall test results and the strength of asphalt mixtures [6].

Siahaya and colleagues studied Asbuton B 5/20 as an additive in AC-WC mixtures to reduce the use of petroleum-based asphalt and improve mixture performance. They used asphalt contents of 5 %, 5.5 %, and 6 %. They used Asbuton B 5/20 contents of 0 % and 2 %. The results showed that the mixture containing 2 % Asbuton B 5/20 with an OAC of 5.5 % met the required Marshall criteria, achieving a stability of 1,655.2 kg, a flow of 3.27 mm, and a VIM of 4.69 %. These findings show that Asbuton B 5/20 has potential as an additive in asphalt mixtures [7].

Setiawan and Rahman studied Asbuton Type 15/25 at 7% and 8% contents in AC-BC mixtures using Pen 60/70 asphalt. Their findings showed that adding 8% granular Asbuton led to a Marshall stability of 1,563.27 kg and a Marshall Immersion value of 82.45%. These results show stability and improved resistance to water immersion compared to the mixture with 7% Asbuton. This confirms that granular Asbuton has the potential to enhance the performance of AC-BC mixtures [8].

Previous studies show that granular Asbuton and epoxy resin can make asphalt mixtures better. Few studies have mixed granular Asbuton and epoxy resin in AC-BC mixtures. In particular few have used epoxy resin to replace part of petroleum-based asphalt. Checked how the mixture holds up under tensile strength. That is why this study creates a mixture that includes granular Asbuton and epoxy resin. In this mix epoxy resin works as a modifier. Also replaces part of the petroleum-based asphalt. What is new here is the pairing of Asbuton and epoxy resin and the way we check the mix. We look at asphalt properties Marshall test results and Indirect Tensile Strength (ITS). We expect this method to give AC-BC mixtures mechanical strength. It also makes use of Asbuton, which is a nearby sustainable material, for pavement.

2. Research Method

This study used an approach carried out at the Transportation Systems and Engineering Laboratory, which is part of the Department of Civil Engineering, Faculty of Engineering Halu Oleo University. The AC-BC hot mix asphalt mixture included key materials. These were Buton Asphalt (Asbuton) Type B 50/30, asphalt with a 60/70 penetration grade, coarse and fine aggregates, mineral filler and epoxy resin. The aggregates and the mineral filler were sourced from Moramo stone materials, supplied by UD. Maju Stone Crusher in Moramo. The mixture was designed using an Optimum Asphalt Content (OAC) of 6.1% and the optimum epoxy resin content was set at 3% based on research conducted by La Ode Muhammad Ally Rahman Rere. The test specimens were cylindrical in shape with a diameter around 10.1 cm and a height of 6 cm. Each specimen had an estimated material weight of, about 1,200 grams. After preparation these specimens were tested in the laboratory to assess the performance and mechanical properties of the AC-BC mixture that included both Asbuton and epoxy resin.

3. Results and Discussion

3.1. Aggregate, Granular Asbuton, Asphalt and Epoxy Resin Characterization Results

In this study the aggregate used was Moramo stone, taken from Moramo District, South Konawe Regency. The other materials included Asbuton and 60/70 penetration-grade asphalt both taken from the Transportation Systems Laboratory, Faculty of Engineering Halu Oleo University. Before being used in the AC-BC mixture the materials went through characterization tests to check their engineering properties and to make sure they met the specifications that were, in place. The test results showed that the aggregate granular Asbuton and 60/70 penetration-grade asphalt met the requirements of the 2018 Bina Marga specifications. The results of the material characterization tests are shown in the following table.



Figure 2. Epoxy Resin

Table 1. Aggregate Characterization Test Results

Type Of Test	Units	Method	Specification	Result	Remarks
Coarse Aggregate Specific Gravity and Water Absorption Test					
Bulk Specific Gravity	-	SNI 03-1969-2008	≥2,5%	2,76	Meets the Specification
SSD Specific Gravity	-			2,68	Meets the Specification
Apparent Specific Gravity	-			2,71	Meets the Specification
Water Absorption	-			1.01	Meets the Specification
Fine Aggregate Specific Gravity and Water Absorption Test					
Bulk Specific Gravity	-	SNI 2531:2015	≥2,5%	2,82	Meets the Specification
SSD Specific Gravity	-			2,72	Meets the Specification
Apparent Specific Gravity	-			2,67	Meets the Specification
Water Absorption	-			1,98	Meets the Specification
Filler Specific Gravity					
Bulk Specific Gravity	-	SNI 03-2531-1991	≥2,5%	2.79	Meets the Specification
Apparent Specific Gravity	-			2.79	Meets the Specification
Aggregate Abrasion Test Using the Los Angeles Machine					
Los Angeles Abrasion Value (LAV)	%	SNI 03-2417-2008	≤40%	33,08	Meets the Specification
Aggregate Silt Content					
Coarse Aggregate Slit Content	%	SNI 03-4142-1996	< 5%	0.82	Meets the Specification
Fine Aggregate Slit Content	%	SNI 03-4142-1996	< 5%	4,14	Meets the Specification

Table 2. 60/70 Penetration-Grade Asphalt Characterization Test Results

Type Of Test	Units	Method	Specification		Result	Remarks
			Min.	Max.		
Penetration at 25°C	Mm	SNI 03-3639-2002	60	70	65,6	Meets the Specification
Specific Gravity	-	SNI 2441:2011	1	-	1,06	Meets the Specification
Ductility at 25°C	Cm	SNI 2432:2011	100	-	147,5	Meets the Specification

Table 3. Granular Asbuton Characterization Test Results

Type Of Test	Units	Method	Specification	Result	Remarks
Abuton Bitumen Content	%	RSNI M -05- 2004	Min. 20%	23,70	Meets the Specification
Moisture Content	%	SNI 2490:2008	Max. 4%	1,31	Meets the Specification

Table 4. Epoxy Resin Characterization Test Results

Type Of Test	Units	Method	Result
Specific Gravity	-	SNI 2441:2011	1,20

3.2. AC-BC Mixture Gradation Design

After the aggregate characterization tests were completed, the next step was to design the gradation. In this study, the gradation was developed using the target method. This method means the gradation was set according to the range that is given in the relevant specifications.

Table 5. Aggregate Sieve Gradation for AC-BC Mixtures

Shieve Number		Type Of Materials	Specification for Percent Passing (%)		Target Percentage Retained	Target Asbuton Mineral Percentage	Moramo Aggregate Material	Percentage
Mm	Inch		Range	Target Gradation	%	%	%	%
19	3/4"	Coarse Aggregate	90 - 100	100	0,0	-	-	64.4
12.5	1/2"		75 - 90	82,5	17,5	-	18,6	
9.5	3/8"		66 - 82	74	8,5	-	9,1	
4.75	No. 4		46 - 64	55	19,0	-	20,2	
2.36	No. 8		30 - 49	39,5	15,5	-	16,5	
1.18	No. 16	Fine Aggregate	18 - 38	28	11,5	5,7	11,7	30.4
0.600	No. 30		12 - 28	20	8,0	7,6	7,8	
0.300	No. 50		7 - 20	13,5	6,5	14,4	5,6	
0.150	No. 100		5 - 13	9	4,5	15,1	3,5	
0.075	No. 200		4 - 8	6	3,0	16,5	1,7	
Filler	PAN	Filler	- - -		6,0	13,9	5,2	5.2

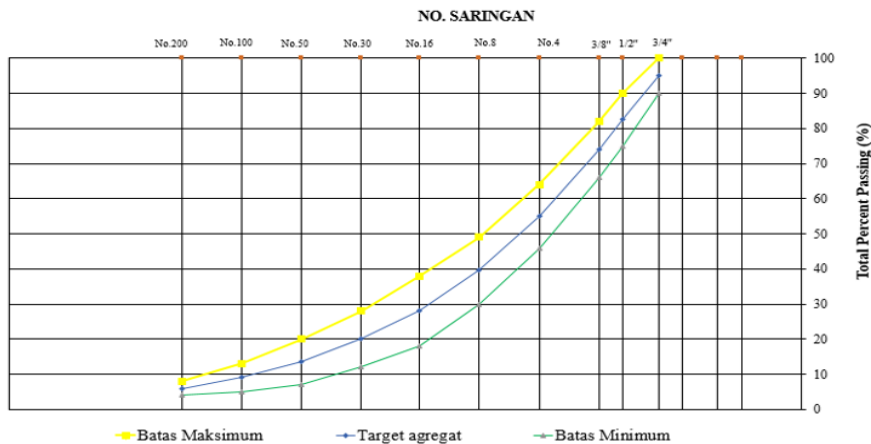


Figure 3. The aggregate gradation curves

The aggregate gradation curve shows how sieve size relates to the percentage of aggregate passing. I see that the target gradation is always between the lower limits for every sieve size, so the designed aggregate gradation meets the AC-BC gradation requirements. The percentage of passing goes up when the sieve size goes down, showing the shift from coarse to fine aggregate fractions. The smooth spread of sizes should give good particle interlocking and a dense mix, which will help the AC-BC mixture perform well.

3.3. Marshall Characteristics Test Results

In Indonesia, we look at the Marshall characteristics of asphalt mixtures using the 2025 General Specifications of Bina Marga. Those specifications give us the rules we need to judge how good asphalt mixtures are. In this study, we use those rules to check Marshall parameters. Those parameters are Marshall stability, flow, VIM (Voids in Mixture), VMA (Voids in Mineral Aggregate), VFA (Voids Filled with Asphalt), and

Marshall Quotient (MQ). We then compare the results from our tests with the rules to see if the AC-BC mixture we made meets the standards and is suitable for use.

Table 6. 2025 General Specifications of Bina Marga

Mixture Properties	Asphalt Concrete (AC)	
	Asphalt Concrete-Binder Course (AC-BC)	
Number of Blows per Face		75
Ratio of Particles Passing the 0.075 mm Sieve to Effective Asphalt Content	Min.	0,6
	Max.	1,2
Voids in Mixture (VIM) (%)	Min.	3,0
	Max.	5,0
Voids in Mineral Aggregate (VMA) (%)	Min.	14
Voids Filled with Asphalt (VFA) (%)	Min.	65
Marshall Stability (kg)	Min.	800
	Min.	2
Flow (mm)	Max.	4
	Min.	90
Residual Marshall Stability (%) after 24-hour Immersion	Min.	2
Voids in the mix (%) at refusal density	Min.	

Based on the requirements listed in the 2025 General Specifications of Bina Marga the Marshall characteristics of the AC-BC mixtures were later studied through lab testing. The test was done to see how adding epoxy resin affected the mixtures in terms of volume and strength. The parameters studied were Marshall stability, flow, voids in the mixture voids in mineral aggregate voids filled with asphalt and Marshall Quotient. The results from the Marshall test gave values for these parameters, for the AC-BC mixtures that had amounts of epoxy resin. The amounts were 0.00%, 1.25%, 2.50%, 3.75% and 5.00%. These results were then checked against the required standards to see if the AC-BC mixtures made in this study met the rules and were good to use.

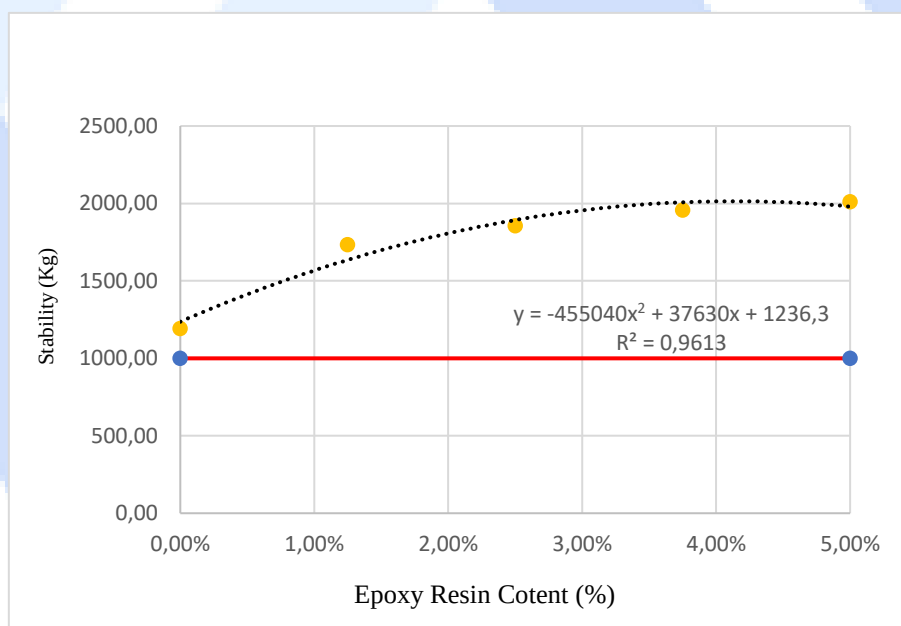


Figure 4. Effect of Epoxy Resin Content on Marshall Stability

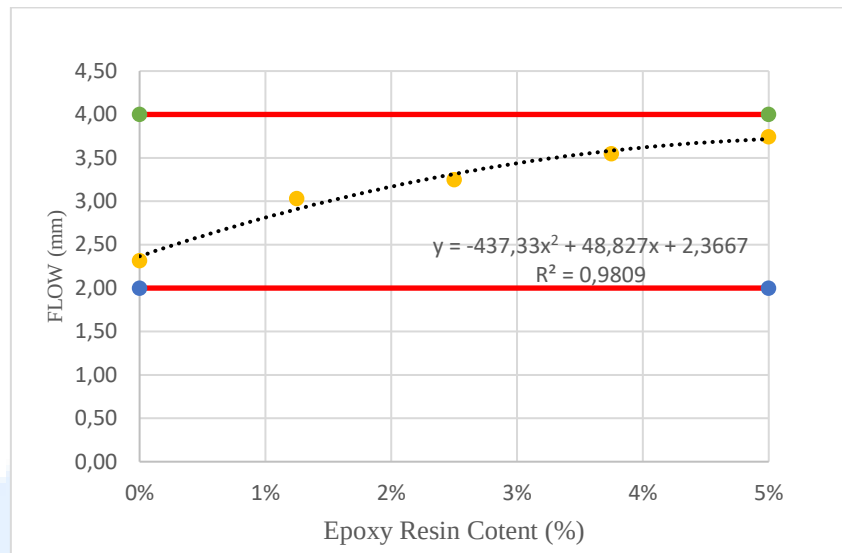


Figure 5. Effect of Epoxy Resin Content on Marshall Flow

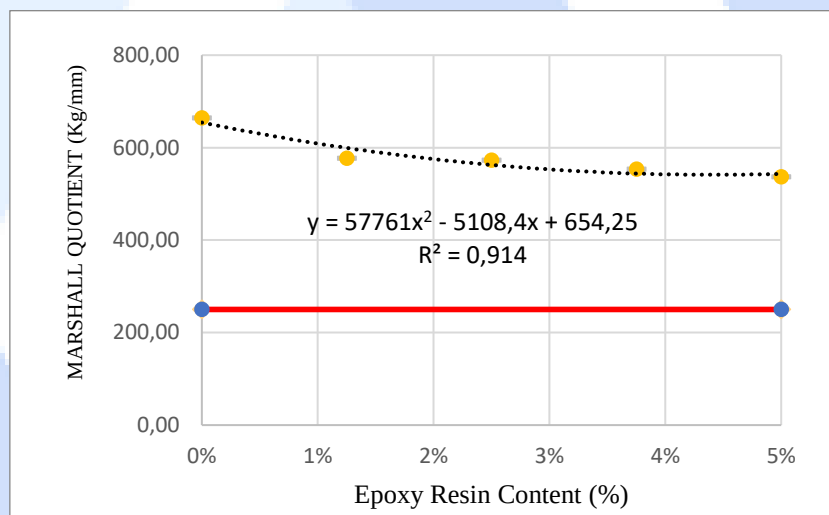


Figure 6. Effect of Epoxy Resin Content on Marshall Quotient

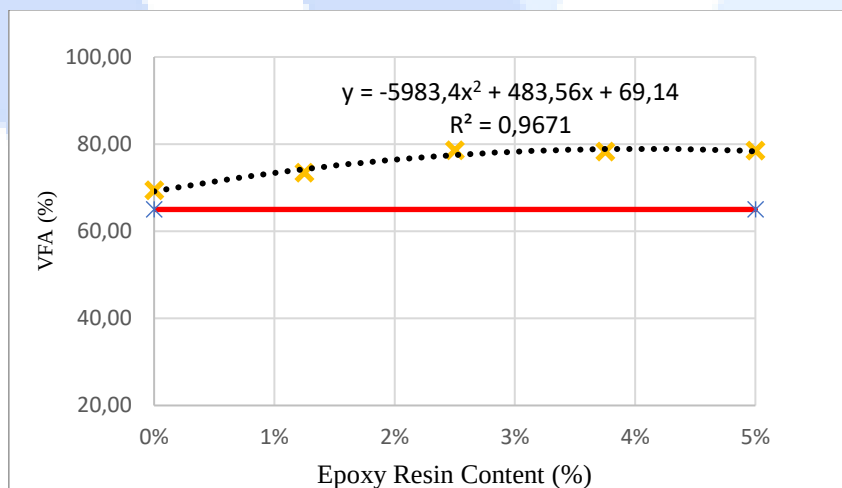


Figure 7. Effect of Epoxy Resin Content on Voids Filled with Asphalt

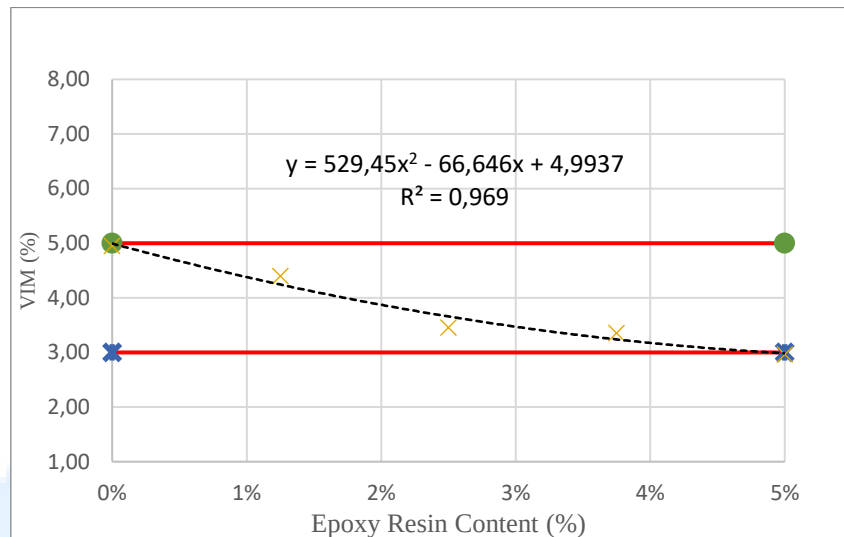


Figure 8. Effect of Epoxy Resin Content on Voids in Mixture

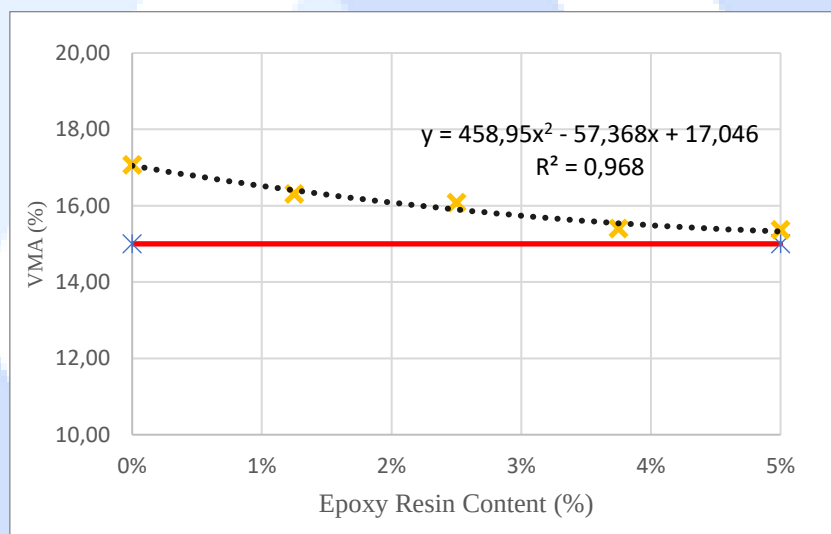


Figure 9. Effect of Epoxy Resin Content on Voids in Mineral Aggregate

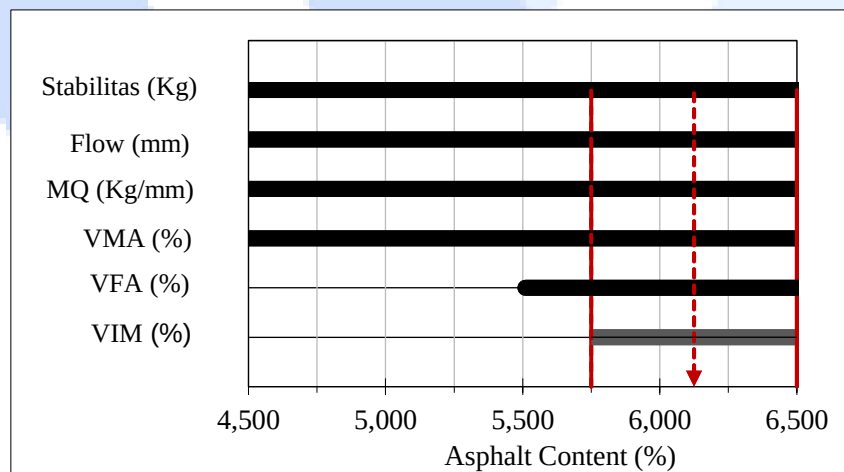


Figure 10. Determination of Optimum Asphalt Content (OAC)

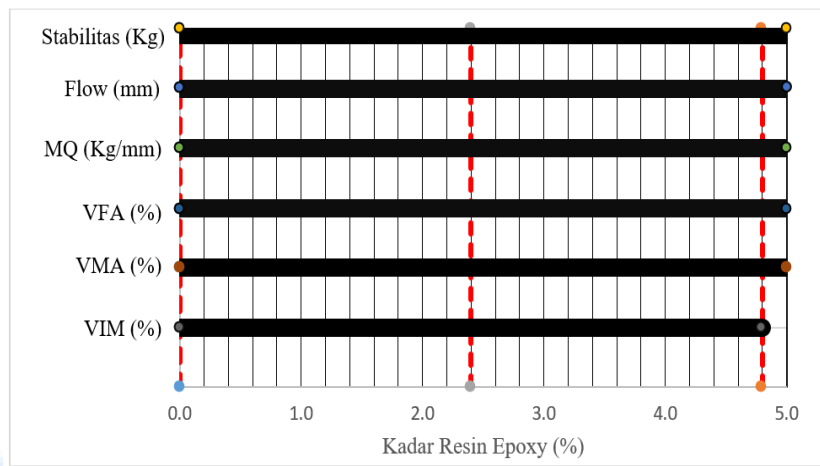


Figure 11. Determination of the Optimum Epoxy Resin Content

Based on the test results, the relationships between Marshall stability, flow, Marshall Quotient (MQ), VMA (Voids in Mineral Aggregate), VFA (Voids Filled with Asphalt), and VIM (Voids in the Mix) were studied to find the asphalt content (OAC) of the AC-BC mixture. For the mixture that used aggregates without epoxy resin, the optimum asphalt content was found to be 6.1 percent. At the same time, based on the test results of the hot-mix AC-BC mixture that used petroleum-based asphalt modified with epoxy resin, the optimum epoxy resin content was found to be 2.4 percent based on the overall Marshall characteristics.

The two graphs show a difference in the optimum content range between the mixtures with and without epoxy resin. The AC-BC mixture that had epoxy resin showed a wider optimum content range, about 0 to 4.8 percent. The AC-BC mixture that did not have epoxy resin had an optimum asphalt content range of about 5.75 to 6.25 percent. This difference shows that adding epoxy resin changes how the asphalt binder behaves. This change then affects the Marshall characteristics of the AC-BC mixture. The wider range seen in the epoxy-modified mixture means there is change in how the mixture behaves when the epoxy resin content changes. The regular mixture has a tight optimum asphalt content range.

3.4. Indirect Tensile Strength Test Results

The Indirect Tensile Strength (ITS) test was conducted to evaluate the tensile resistance of the AC-BC mixture and to provide an indication of its susceptibility to cracking in the field, particularly bottom-up cracking at the bottom of the binder course layer. A comparison of the Indirect Tensile Strength (ITS) of the AC-BC mixtures after 2 days of conditioning and without conditioning is presented in Figure 12.

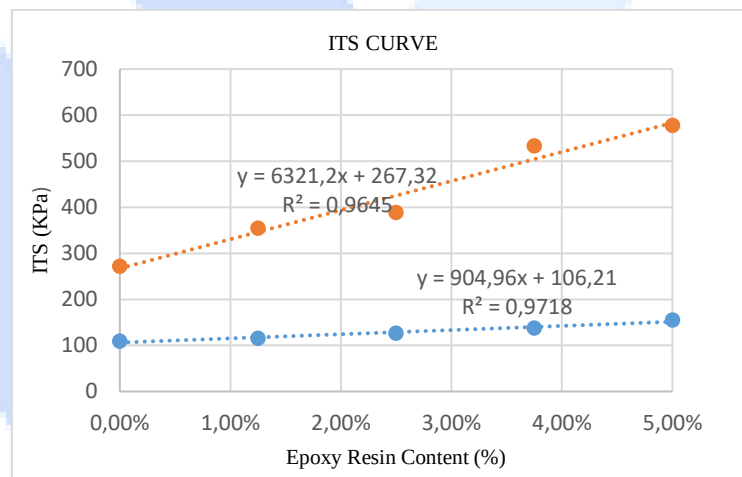


Figure 12. Combined ITS Graph of AC-BC Asphalt Mixtures

Based on the summarized Indirect Tensile Strength (ITS) results under the condition the highest average ITS value was obtained from the AC-BC mixture containing 5% epoxy resin with an ITS value of 155.00 kPa. In contrast the lowest average ITS value was observed in the mixture without epoxy resin (0%) with an ITS of 109.29 kPa. Under the 2-day conditioning condition the highest average ITS value was also obtained from the mixture containing 5% epoxy resin reaching 577.94 kPa while the lowest average ITS value was obtained at 0% epoxy resin with an ITS of 271.98 kPa. Overall, the results indicate that ITS increased with increasing epoxy resin content. The blue line represents the specimens while the red line represents the specimens conditioned for 2 days. The ITS values after 2-day conditioning were consistently higher than those of the specimens across all epoxy resin contents indicating an improvement, in the tensile resistance of the AC-BC mixtures following the conditioning period.

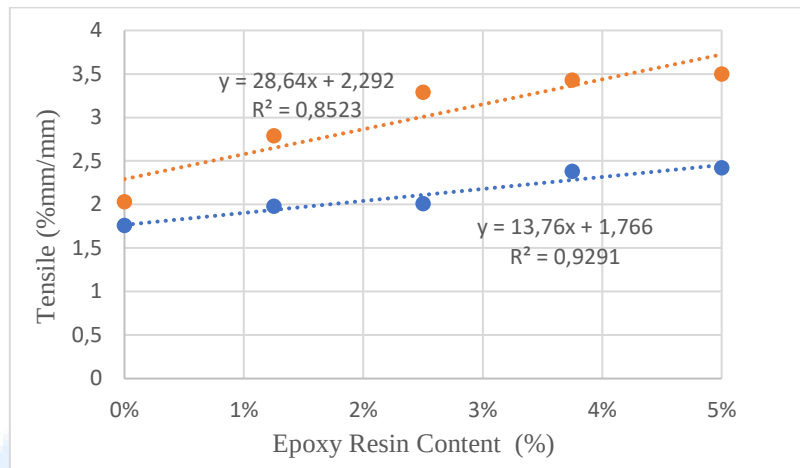


Figure 13. Combined Strain Graph of AC-BC Asphalt Mixtures

The graph shows how the amount of epoxy resin affects the tensile strain of AC-BC asphalt mixtures under two conditioning conditions. The results show that tensile strain goes up as the epoxy resin content goes up. The blue line stands for the specimens and the red line stands for the specimens that were conditioned or immersed for two days. For the specimens tensile strain rose from about 1.76 at 0% epoxy resin to 2.42 at 5% epoxy resin. The coefficient of determination is $R^2 = 0.9291$, which shows a positive link between epoxy resin content and tensile strain. For the specimens conditioned for two days the increase in tensile strain was both higher and steeper. It went from around 2.03 at 0% epoxy resin to 3.50 at 5% epoxy resin with an R^2 value of 0.8523. The regression equations are $y = 13.76x + 1.766$ For unconditioned specimens and $y = 28.64x + 2.292$ for the 2-day conditioned specimens. These equations support the relationship between epoxy resin content and tensile strain. The steeper slope for the 2-day conditioned specimens means their tensile strain responded strongly to increases in epoxy resin content. At 5% epoxy resin the tensile strain under the 2-day conditioning condition was 44% higher, than that of the unconditioned specimens. This suggests that adding epoxy resin improves the ability of the AC-BC mixture to deform under tensile stress after two days of conditioning. This improvement may help the mixture resist tensile cracking. Make it more flexible.

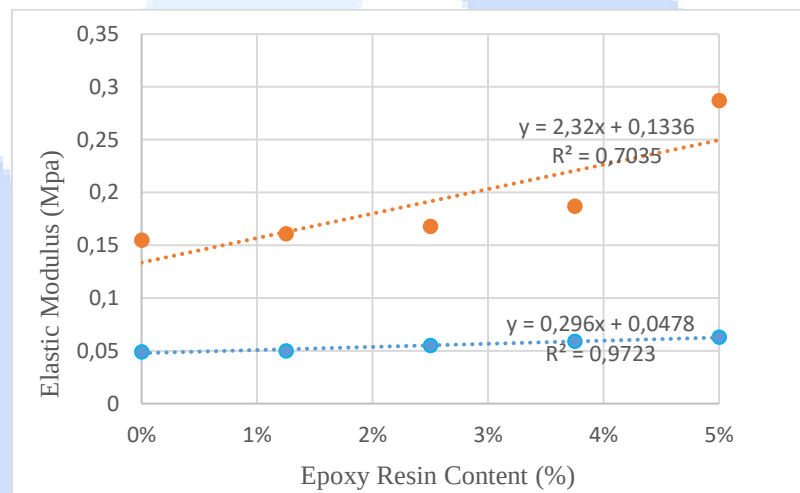


Figure 14. Comparison of Elastic Modulus Values for AC-BC Asphalt Mixtures

The graph shows how the elastic modulus of AC-BC asphalt mixtures changes with amounts of epoxy resin under two conditions. One condition is specimens and the other is specimens that were immersed for two days. In both cases the elastic modulus goes up as the epoxy resin content increases. For specimens, the elastic modulus starts at about 0.048 MPa when there is no epoxy resin. It rises to around 0.065 MPa when the epoxy resin content reaches 5%. This relationship follows a line with the equation $y = 0.296x + 0.0478$. The R^2 value is 0.9723, which's very close to 1. This high R^2 value means there is a clear linear relationship between the amount of epoxy resin and the elastic modulus. For specimens conditioned for two days the increase in modulus is much stronger. The modulus starts at 0.134 MPa with no epoxy resin and increases to around 0.290 MPa at 5% epoxy resin. The regression line here is $y = 2.32x + 0.1336$, With an R^2 of 0.7035. This shows a to strong positive relationship. The slope of the line is much steeper compared to the specimens, meaning the elastic modulus becomes more sensitive to changes in epoxy resin content after the conditioning period. At 5% epoxy resin the elastic modulus for conditioned mixtures is 346% higher than for unconditioned mixtures. This large difference shows that the two-day conditioning significantly boosts the stiffness of the mixture. Overall adding epoxy resin increases the stiffness and elastic response of the AC-BC mixture. The rise in modulus suggests that epoxy resin improves the bonding between binder and aggregate. This leads to load transfer and greater resistance to deformation under tension. The higher modulus after conditioning also shows that the interaction between binder and aggregate strengthens over time. This improved interaction makes the mixture stiffer.

However, increasing stiffness must be balanced with flexibility. Much stiffness might make the mixture more prone to cracking under stress. So, while the increase, in modulus is positive it is important to look at tensile strain results as well. The goal is to find a balance where the mixture's stiff enough to resist deformation but flexible enough to avoid cracking.

4. Conclusion

- The results from the modified asphalt characterization tests show that adding epoxy resin changes the engineering properties of 60/70 penetration-grade asphalt. As the amount of epoxy resin increases the penetration value goes up. The elastic behavior of the asphalt binder improves. These findings come from penetration and ductility test results. The Thin Film Oven Test (TFOT) also gives clues. The lowest weight loss happened when the epoxy resin content was 3.75% while the highest weight loss occurred in the mixture with no epoxy resin all (0%). This means that including epoxy resin helps the asphalt binder resist mass loss during short-term aging.
- The Marshall test results for the 60/70 penetration-grade asphalt modified with epoxy resin in a hot-mix AC-BC mixture that includes Asbuton were tested with different epoxy resin levels: 0%, 1.25%, 2.50%, 3.75% and 5.00%. These results reveal shifts in both mechanical traits of the mixture. Marshall stability, flow and VFA generally rise as more epoxy resin is added. However, the Marshall Quotient (MQ) VMA and VIM tend to fall. These changes suggest that epoxy resin alters how the binder interacts with the aggregate and how the structure forms. Higher stability means load-bearing ability. Meanwhile the drops in VIM, VMA and VFA point to changes in the makeup and how well the asphalt fills the spaces between aggregate particles.
- The Indirect Tensile Strength (ITS) test was performed on the 60/70 penetration-grade asphalt modified with epoxy resin in the hot-mix AC-BC mixture containing granular Asbuton. The epoxy resin amounts used were 0%, 1.25%, 2.50%, 3.75% and 5.00%. Testing was done under two conditions: specimens and specimens soaked for two days. In every case the ITS values were higher for the specimens after two days of conditioning. This improvement shows that the soaking period allowed bonding between the epoxy-modified asphalt binder and the aggregate. This leads to greater tensile resistance. The peak ITS value came at 5% epoxy resin content. So increasing the epoxy resin level generally makes the AC-BC mixture stronger, in tension.
- After looking at all the data—on asphalt properties, Marshall performance and ITS results—the best epoxy resin amount was set at 2.4%. The optimum asphalt content (OAC) was found to be 6.1%. Combining Asbuton with epoxy resin looks promising for improving the quality of AC-BC mixtures. It also supports using Asbuton as a pavement material, which reduces reliance on regular petroleum-based asphalt.

Declaration of Conflict of Interests

The authors declare that they have no known competing financial or non-financial interests that could have influenced the work reported in this paper.

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Increasing the Robustness of Concrete and Steel Bridges for the Effect of Collision

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Keywords

Robustness,

Pier failure,

Force equilibrium,

Effect of collision.

Abstract

Current regulations mandate that structures be designed with adequate robustness, which can be achieved by adding more resistance to members or by developing other load paths. The latter routes appear to be an impractical solution in the event of a column pier demolition after collision since another column or bracing cannot replace the loss of a crucial support. A concrete bridge's adjacent beams are out of balance if all of the pier's columns are demolished, unless they become continuous, in which case a commensurate bending moment may occur. Two forces can be used to deliver the latter. The bridge slab itself provides the compressive force on top of the beams. A steel plate positioned beneath the adjacent beams' bearings can supply the tensile force. A simple experiment has enabled to validate this idea. Two examples of concrete bridges and one composite bridge have been used to test this theory. A typical road bridge of two 22.5 m span with traditional I-section girders and a 34 m span bridge with 39 precast prestressed hollow core girders. In order to counteract the effect of bonded tendons becoming inactive at the extremities of the PC-beams, dummy cables are required. While there are still some details to be worked out, the overall concept seems to be working and could satisfy the robustness criterion.

1. Introduction

Road accidents are regrettable, yet they appear to be unavoidable. Sometimes the consequences of such accidents are serious and people are heavily injured or even worse. From a structural point of view, bridge collapse due to collision is a worst case and accidental scenario. However, it does happen and total collapse can be the ultimate result. The consequence of total collapse of a bridge superstructure is catastrophic, if the subjacent traffic is hit by the falling bridge deck. In [1] the causes of bridge collapses have been researched. The graph of figure 1 [1] exhibits the results of this survey, found by various researchers. Obviously, the result depends on the local traffic conditions and safety measures, yet impact seems to be the third cause of bridge collapse.

Figure 2 illustrates the result of a collision that severely destroyed a pier. On June 30, 2007, this collision happened on the road that circles Brussels. The pier's transverse head beam was left cantilevered after the disaster destroyed two of its five columns (2007) [2]. Three of the beams in the PC superstructure were lowering as a result of the beam's raga sag. Fortunately, there was a membrane action in the bridge slab that kept the beams from collapsing onto the motorway. In the event that the pier is partially destroyed, this membrane activity could provide an alternate load path. Should the remaining columns not have additional resistance and fail, total destruction would have resulted, since the membrane action cannot be activated. This hypothesis is adopted in this study. Even more well-known is the May 14, 2004, disaster that severely damaged a pier of the Tancanhua Street Bridge in Corpus Christi, as illustrated in (2004) [3].

The unintentional removal of a structural element is discussed in Eurocode EN 1990 par 2.1 (2003) [4] since it may only cause local damage and not a structural element's overall failure. According to Annex A of EN 1991-1-7 [5], buildings may sustain only minor damage in an unintentional incident before collapse. The consensus is that, even though these standards do not define robustness precisely, it should be made sure that, in the event a significant building component becomes unsuitable, progressive collapse does not take place. Although there can be harm, it is crucial to make sure that no lives are lost. Allowing the forces to transmit through alternate paths is one method to demonstrate robustness. For building frames, floors, walls, and other statically indeterminate building components, this is quite straightforward to visualize. Reports on tests and simulations of heavy vehicle collisions on piers for bridges [6] and a protocol for trucks carrying sand have been produced. In [7], the insufficiency of existing design approaches is acknowledged and a pragmatic approach is proposed.

Measures should be put in place to help reduce the overall effects of component failure. According to [8] physical measures should be implemented to reduce failure of components. It is advised to perform vulnerability assessments based on scenarios of vehicle collisions [9]. However, there is no chance of finding an alternative load path in the case of a concrete bridge pier supporting two parallel simply supported

beams from two neighboring spans. If one imagines the mechanisms that emerge when an entire bridge pier vanishes, it is possible to construct a support, although one with limited strength.

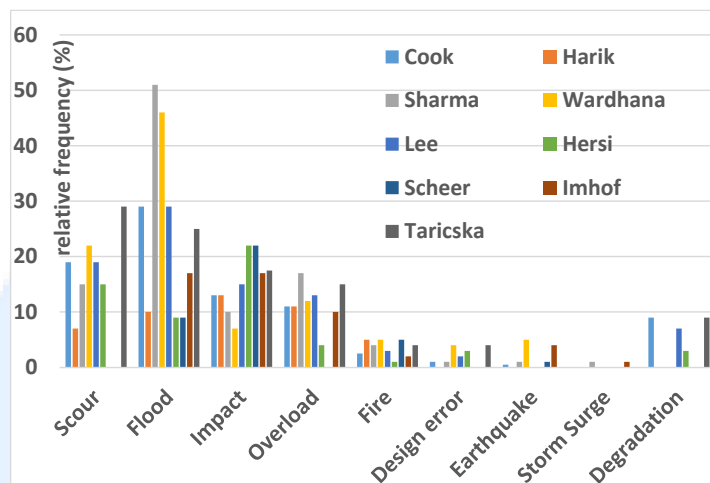


Figure 1. Frequency of bridge collapse causes



Figure 2. Damaged pier after truck collision

2. Equilibrium of forces

A typical arrangement of two ends of adjacent bridge decks with PC beams is shown in figure 3 left. It can be seen that both spans are independent and are connected only by a bridge joint in the road pavement and that the independent supports of both sides are mounted on a head beam of the pier.

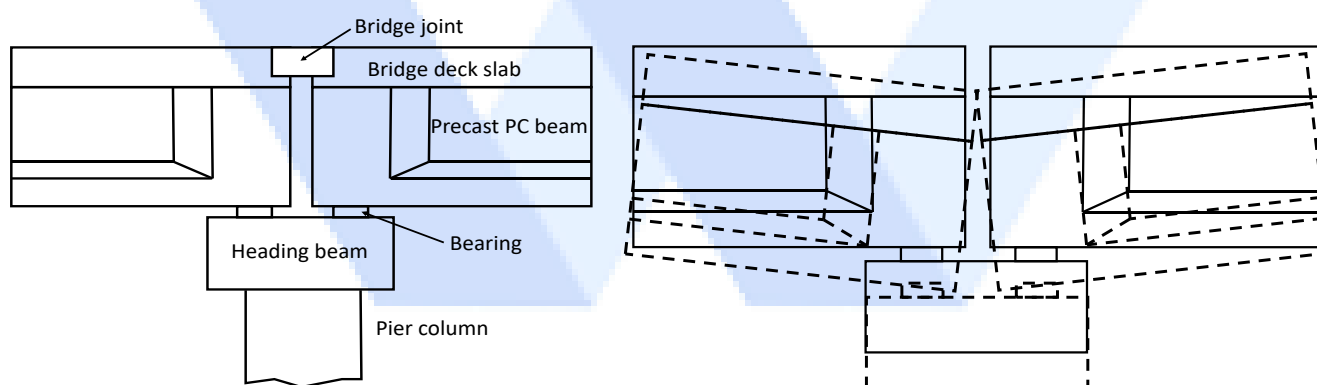


Figure 3. Joint between two adjacent bridge spans. (Left initial right pier collapsed)

The bridge joint may possibly be replaced by a so-called flexible concrete slab. If the support point of the pier disappears, the head beam will also tend to fall, or at least to descend. The two ends of the adjacent bridge decks therefore descend and try to assume a rotated position, as indicated in dotted line in figure 2 right. At the top, the two bridge decks will bump against each other and a compressive force can therefore arise. If global collapse is to be avoided, then it is sufficient to generate a tensile force at the bottom, for example at the level of the bearings. Together with the compressive force at the top, this tensile force forms a torque that may be able to withstand a new bending moment. The forces are exhibited in figure 3. This bending moment arises because the supporting point of the pier has disappeared, as a result of which the span of the beams has become equal to the sum of the spans of the two adjacent bridge decks.

The compression force in the bridge deck slab is a localized force. As a result, according to EN 1992-1-1 (2005) a fairly high pressure can be allowed. Therefore, this condition is not a real problem. To prevent collapse, 3 conditions must be fulfilled. The first condition is that the head beam must not fall. Simple means can be designed for this purpose, such as the installation of flexible tension rods that fix this beam to the bridge deck slab or the girders. Naturally, these tie rods must have sufficient play, so that sufficient lowering of the coupling beam can take place to allow the two bridge deck plates to butt against each other.

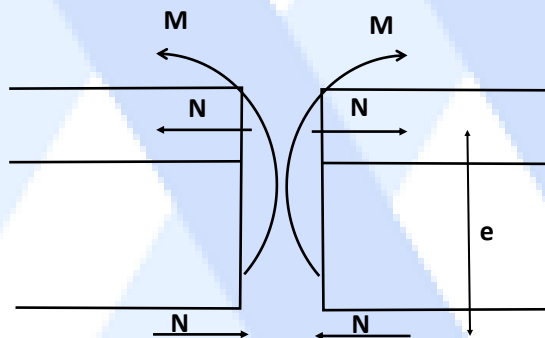


Figure 4. Force equilibrium after pier collapse

The second condition is that the tensile force is supplied. Several methods are possible for this. The first method consists of supplementing the bearings with a longitudinal stop block, which allows sufficient play during normal movements, but again blocks the support in the accidental state. The support device must then be provided with a steel base plate, which is rigidly attached to the coupling beam. This steel base plate must be common to corresponding bearings of the girders of both adjacent bridge decks.

A second method is similar to the first, but the tensile force is transferred between the adjacent beams by means of the coupling beam itself. This probably requires heavier anchoring of the supports and a considerable increase of the normal transverse reinforcement of the coupling beam. Of course, one can choose to combine the first and second method.

The third method consists of providing a 'passive' tension element. This tendon may consist of an untensioned prestressing cable or similar. This is very similar to the reinforcements that can be applied to the PC beams.

3. Demonstration

The above principle was demonstrated by a simple experiment, which had no other purpose than to demonstrate its possibility. The arrangement is indicated in figure 4. Two adjacent PC beams have been replaced by heavy steel SHS profiles 60/5, each resting with a roller bearing on a steel plate, which will play the role as described in method 1 above.

The profiles are also connected to the steel plate below the bearings by tie rods, which pass through oversized holes in the profiles, to allow free rotation of the profiles. As can also be seen in figure 5, stop strips are provided to limit the horizontal movement of the roller bearings. At the top of the steel profiles, a plastic plate is placed between the edges. This represents the effect of a tightly pressed bridge joint.

In figure 5 the support point is removed. The entire arrangement has sunk, except for the opposite ends of the profiles, which rest on stones. The roller bearings have moved against the stops. The profiles have not fallen, because they are held in place by the tie rods. It is important that the tension plate has functioned and can compensate for the pressure on the plastic plate.

Table 1. Characteristics of demonstration test

Condition	Span (mm)	Weight	W (mm ³)	M (Nm)	σ (MPa)
Support active	790	8.42 N/m	17.75 10^4	0.657	0.0037
No support	1580	8.42 N/m	28615	2.627	0.0918

No measurements were taken during this demonstration, because the sole purpose was to demonstrate the workability of the principle. Yet it is useful to find out which are the modifications of the forces. Table 1 provides an overview.

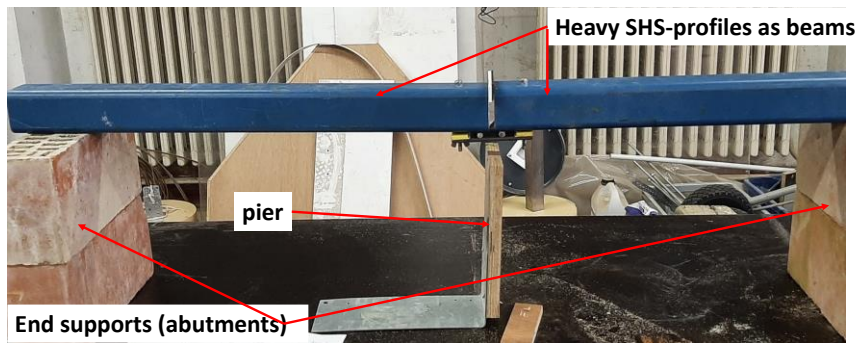


Figure 5. Experimental setup

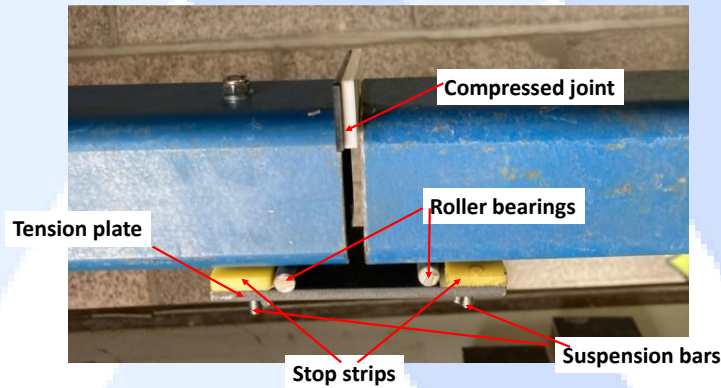


Figure 6. Detail of experiment

As long as a support point exists, the largest tension stress in the profiles can be found halfway their span. Taking into account the moment of resistance, the bending stress is given in the last column of table 1. As soon as the support point has disappeared, a beam with doubled span and thus four times bending moment is created, now at the cross-section where the support point used to be. In that cross-section, the moment of resistance is much smaller, because only the upper flange and the tension plate can provide the torque of forces. It can be seen in the last column that the steel stress has increased 28 times. The corresponding compressive and tensile forces are 27.54 N. It is clear that the force effects are considerable. The concept as such may have been demonstrated, but whether it can be realized in practice is uncertain. Clearly the need exists to examine in concrete cases whether the principle can also be realized in practice.

4. Application to Real Bridges

The first example is an ecoduct with 3 spans of 34.40 m over the Antwerp-Rotterdam motorway and the parallel railway high speed line. It is presented in figure 6. This is not a road bridge, but a 50 m wide ecoduct, providing a safe crossing point for wildlife, whether large or small.



Figure 7. Ecoduct

The main load is 0.50 m of wet soil, plants and low-growing trees. The superstructure consists of 3 * 39 precast PC girders with bonded strands. The cross-section is U-shaped and these beams are almost in contact next to each other. If a pier disappears, then this is clearly an accidental

condition. According to EN 1991-1-7 (2010), for such condition the combination of equation (1) must be used. There are no traffic loads in this comparison, nor an accidental load. It is the static state that makes up the accidental situation.

$$\sum G_k + P + A_d + \psi_{11} Q_{k1} + \sum \psi_{2i} Q_{ki} \quad (1)$$

In this case the compressive force at the top edge of the beams has a value of 61.5 MPa, which is certainly acceptable since $2.5 \cdot f_{ck} = 75$ MPa. The tensile force of 18475 kN can be resisted by a steel S 460 plate fixed below the bearings of $52 \cdot 800$ mm. The neoprene bearings must allow an angular displacement of 0.014 rad and a displacement of 25 mm. The latter is a bit too large and the neoprene height should be 175 mm instead of the available 157 mm. The end of the U-beams must be reinforced with 3 dummy prestressing cables of 27 T15.7 strands (12150 mm²).

The second example is a road bridge for local traffic, crossing the same motorway. It has 4 spans of 23.4 m with PC beams of 1.45 m height. The front view is shown in figure 7. In this case and if a pier is lost, a pressure of 50.9 MPa is created at the top of the cross-section, which is certainly acceptable. A tensile force of 3668 kN is created below the bearings of the same pier, for which a steel S355 tension plate of $30 \cdot 500$ mm is sufficient. To resist the additional bending moment, 14 dummy strands T 15.2 (1972 mm²) are necessary.

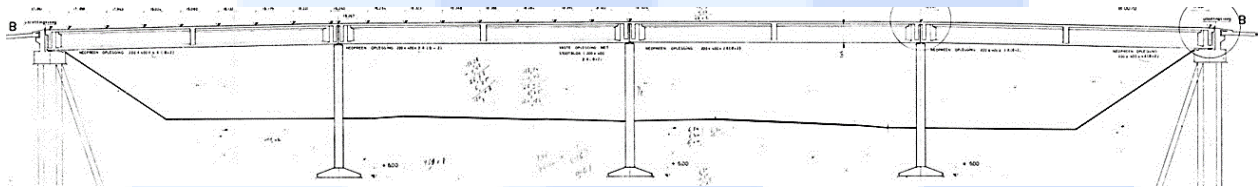


Figure 8. Local traffic road bridge

A third application concerns a railway viaduct for double track, consisting of 4 steel I-beams supplemented with a top concrete slab. Part of the viaduct, is exhibited in figure 9 left and a schematic cross section is illustrated in figure 9 right. Each span equals 59.62 m.



Figure 9. left part of composite railway viaduct, right schematic cross section single beam

If one pier fails, continuity at the joint may be achieved by the method of supplementing the bearings with a longitudinal stop block. The lever arm of the tensile force then reaches 3.45 m, thus including the height of the bearing. Assuming no traffic will be present after pier failure, the total bending moment to be resisted is due to the dead weight (14.44 kN/m), the concrete (20 kN/m) slab and the ballast and rails (23.8 kN/m), in total 104169 kNm. The tensile force to be resisted equals 30193 kN for each beam. If the steel plate below the bearings has the same width as the lower flange (780 mm) and thickness of 90 mm, in this accidental situation its stress will reach $30193000 / (780 \cdot 90) = 430$ MPa. Hence, a large deformation will occur, but the superstructure will not fail and fall down on the installation below the viaduct. This demonstrates the robustness achieved.

5. Conclusion and Outlook

It has been attempted to show that, when a pier is completely destroyed, the complete collapse of the superstructure of PC bridges can be avoided by taking some precautions. A couple of forces in the superstructure at the level of the destroyed pier can ensure that a bending moment is resisted, which is certainly larger than in normal condition. It should be taken into account that this is an accidental condition.

The idea of creating the couple of forces has been worked out in outline for 3 practical cases, the first of which is a large bridge. No fundamental problems were found in this elaboration. The use of dummy prestress can be applied to increase the lack of strength of PC beams at the lost pier location, in accidental condition. A small qualitative experiment shows that the idea works and confirms the insight about the feasibility of the proposed solution. In the present state of development of the proposal, it may be suspected for bridge spans above 25m that the dummy prestress will have to be long and must be made active during the calamity situation only, without interfering with the use conditions of the girders. However, the elongation needed to activate the dummy tendons may not be available during the sagging of the demolished pier. In future, further development and experiments are planned to confirm the preliminary conclusions and to further substantiate the feasibility of the concept.

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Various Types of Re-Use of Decommissioned Wind Turbine Towers

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Reuse of steel,

Human induced vibration, Fatigue,

Footbridge.

Abstract

The reuse of decommissioned structures with a short lifespan can contribute significantly to limiting greenhouse gases and CO₂ emissions. Wind turbines have a relatively short lifespan of around 25 years. This is due to the numerous moving parts, the turbine blades, and the rapidly changing requirements for power and performance. Dismantling wind turbines is labor-intensive, and few components can be reused. The study considers multiple applications for the reuse of the towers. Among the various possibilities, use as a pedestrian bridge with a semi-circular cross-section appears the most promising. This latter application requires that multiple conditions be met. In addition to the strength of the steel tower, local buckling must also be avoided, as well as vibration due to wind resonance, vibration due to human-induced effects, and the fatigue strength of the footbridge. Regarding the latter, the fatigue damage incurred during the wind turbine phase combines with that incurred during use as a footbridge. A systematic parameter study into the relationship between diameter and plate thickness for various spans as a footbridge can be helpful in determining approximate solutions.

1. Introduction

As the impact of global warming intensifies, increasing sustainability in the energy sector has become a pressing challenge. One widely adopted solution to reduce greenhouse gas emissions is the generation of renewable energy, particularly wind turbines. Over the past decade, their implementation has increased rapidly, both onshore and offshore.

Despite their environmental benefits, wind turbines undergo high cyclic loads throughout their lifetime, which leads to fatigue and wear of the mechanical components. As a result, the safe operational lifespan of wind turbines is relatively limited, typically around 25 years. Nevertheless, the energy and greenhouse gas payback times are relatively short, approximately 6 months to 1 year [1].

From a sustainability perspective, the decommissioning of the wind turbine should be optimized in order to maximize the re-use, re-purposing and recycling of its components. Steel constitutes a considerable amount of the building materials, accounting for approximately 56 % of the total embodied energy and 35 % of the CO₂ emission of the construction process [1]. Currently, most steel components are recycled despite its disadvantages and difficulties like the dismantling of large tower components. Re-purposing the wind turbine tower may provide a more sustainable end-of-life solution of this component.

The goal of this ongoing research is to determine which types of structures can be designed and built from the towers of decommissioned wind turbines. To this end, additional insight is needed in the composition and life experience of this type of structure.

2. Evolution of Wind Turbines

Wind energy has been used for thousands of years, for example, to sail the seas or grind grain. In the late 19th century inventors began to explore the use of wind to generate electricity. Early pioneers from 1880 to 1900 established key design aspects for wind turbines, recognizing the necessity of automatic speed and power control to ensure reliable operation. This also revealed that turbines with fewer blades perform better aerodynamically.

During the 1970s, the oil crises prompted a global recognition of the need of renewable energy sources. The interest in alternative energy production, including wind energy, increased and the commercialization of both onshore and offshore wind turbines grew. Despite this growth, the wind turbine design coalesced around the standard configuration, with relatively few fundamentally new design concepts [2].

Looking ahead, Europe is expected to install an additional 178 GW of wind power capacity between 2025 and 2030. By 2030, this would lead to 344 GW of wind energy capacity, with 298 GW onshore and 46 GW offshore. Assuming an average power rating of 4 MW and 9 MW for onshore and offshore wind turbines respectively, this corresponds to 74 500 onshore and 5 111 offshore wind turbines. Turbines are also expected to increase in size and height. While the removal trend of wind turbines remains uncertain due to market fluctuations, a stepwise availability of dismantled turbines is anticipated [3].

Wind turbines permit a sustainable generation of electricity from renewable energy resources. However, their materials, production, transportation, usage and disposal still require a substantial use of energy and carbon emissions. The typical life time of a wind turbine is approximately 20 to 25 years. The energy produced during its life time exceeds the energy needed for its construction. However, optimizing the life cycle, especially the end of life of wind turbines, can greatly contribute to a higher sustainability of wind turbines.

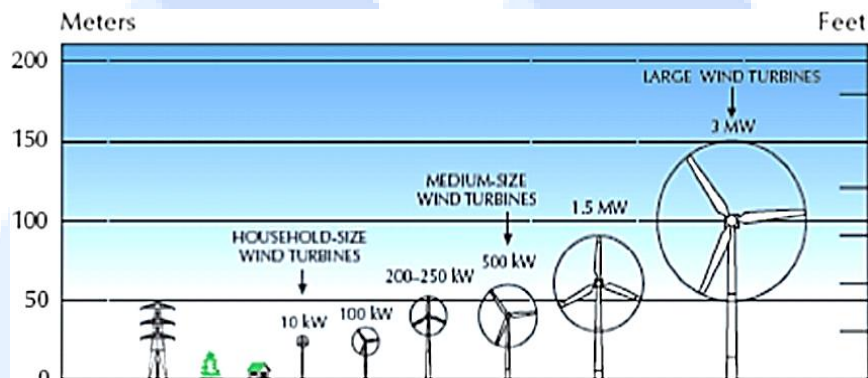


Figure 1. Classification of wind turbines by size and power

When a turbine is permanently removed, it is dismantled and taken apart into separate pieces for further processing. Reusing the components should be prioritized, however this is not always feasible due to the fatigue history of wind turbines. Repurposing the components represents a second option and a last resort would be to recycle them. In Europe, the disposal of wind turbine components in landfills is prohibited [4].

Metals account for the largest share of the embodied energy in the construction of wind turbines, with steel being the dominant material. Steel recycling is a well-established industry, recovery rates can reach up to 85 % to 90 %, reducing the CO₂ emission substantially compared to the production of new steel. Nevertheless, recycling steel from wind turbines still includes additional challenges, including difficult disassembly, the large size of components, and transportation over long distances. Repurposing large steel elements may therefore offer a more sustainable alternative [5].

3. Alternatives for Re-Use of Turbine Towers

Because of their geometry and dimensions, steel circular shafts or towers are particularly suited for re-use. The somewhat conical shape may be interesting as a main structural element. In this section, ideas are tested to propose partial re-use of shafts.

3.1. Characteristics of Turbine Shafts

In what follows, an example of a turbine was taken as a guidance. It corresponds to a medium-sized turbine, which was designed according to a pushover study [6]. In this study a series of 9 segments is considered and analyzed by FE-models, including the effects of wind load, dead load, local buckling with imperfections and seismic action. The results are given in terms of the required plate thickness and diameter. Thus, acceptable characteristics have been obtained.

Plate thickness of turbine shafts is generally rather thin. As a consequence, the slenderness exceeds $D/t = 50$. To prevent local buckling, stiffening is required. However, most types of shafts have diaphragm stiffening only and no longitudinal stiffening. Therefore, the diaphragm distance must be sufficiently low. The absence of longitudinal stiffening is exhibited in figure 2, which displays a decommissioned shaft on lorry to transport it to a location of re-use.



Figure 2. Decommissioned shaft for re-use

3.2. Re-Use as Underground Water Tank

Underground water tanks contribute to fresh water collection and saving. Some insight of the feasibility is obtained by considering the case of a 5 m part of the shaft to be buried according to figure 2, exhibiting the most detrimental case of an empty tank loaded by lateral soil and ground water pressure. The largest bending in the retrieved shaft part is found at the bottom of the tank. The stresses at this location are due to the edge bending moment m_0 and the hoop stress. Calculation of the stress state can be found in literature [15]. The equation allowing to determine m_0 is (1).

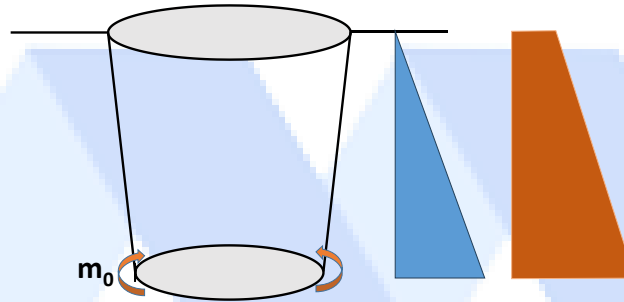


Figure 3. Principle of underground water tank

$$m_0 = -\frac{\gamma}{4} \frac{2(\lambda h - 1) E' t'^3 + 3(1 - \nu') k h r^3}{\lambda^3 E' t'^3 + 6(1 - \nu') k r} \quad (1)$$

In eq. (1) E' denotes the modulus of the lower base plate, t' its thickness, γ denotes the specific weight of the liquid, h is the height of the tank ν' denotes the Poisson coefficient of the base plate material, r is the average radius of the tank and λ is given by equation (2).

$$\lambda = \sqrt[4]{\frac{3(1 - \nu^2)}{r^2 t^2}} \quad (2)$$

In equation (2) ν denotes the Poisson factor of the cylinder material, t the thickness and r the cylinder radius.

If the tank depth equals 5 m, the thickness is varied and the soil pressure is neutral, Table 1 exhibits the bending moment m_0 and the stress due to meridional and hoop effects. In this Table, the diameter of the cylinder equals 3.6 m, the modulus of steel = 210000 MPa, the water density = 10 kN/m³ and density of saturated soil = 20 kN/m³. Since the bottom plate shows a tendency to lift instead of pressing on the subsoil, the soil stiffness of 50000 kN/m³ has but little influence. The radial restraint of the cylinder lower circumference is the main boundary condition. In Table 1, stress has not yet been multiplied by partial safety factors. Therefore, the plate thickness should correspond to 25 mm, for steel grade of $f_y = 355$ MPa.

Table 1. Stresses and bending in buried tank

t (mm)	15	20	22	25	30
m_0 (kNm/m)	27.21	27.47	27.18	26.65	26.66
σ (MPa)	732.7	417.3	341.7	260.1	181.2

According to [7] the requirements correspond to the lower part of the shaft of a 53 m high wind turbine shaft. Hence, sufficient evidence exists to consider re-use as water tank.

3.3. Re-Use as Separate Cellar

Some privately owned houses have no basement. This deprives owners of storage and in particular of dry food, vegetables or wine. Companies have developed circular precast RC cellars for wine storage. The alternative of re-used parts of towers requires similar analysis as for the empty water tank. Inside the cellar additional diaphragms may be necessary to form the shelves for storage. The general idea is exhibited, including the spiral staircase.

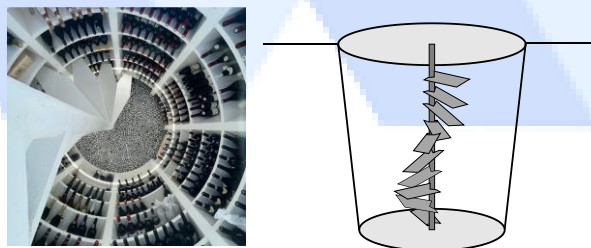


Figure 4. Principle of separate cellar

3.4. Re-Use as Central Core of Multi-Story Building

In multi-story buildings, sufficient resistance for horizontal wind force is obtained by providing central cores. This arrangement is by far more economical than to provide sufficient resistance to columns and beams and construct high-performing frames. In the case of the 5 floor building of figure 5, the beams and columns must also resist the lateral wind force. Staircases are provided at the 4 corners of the building. These might be designed as stiff cores of the building. Although the stairs have a rectangular plan view, this may be transformed to spiral staircases and constructed by re-use of turbine shafts.

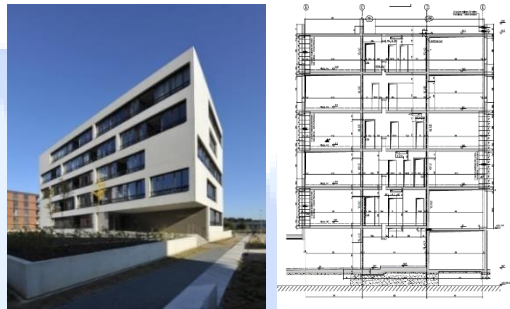


Figure 5. A five-floor building

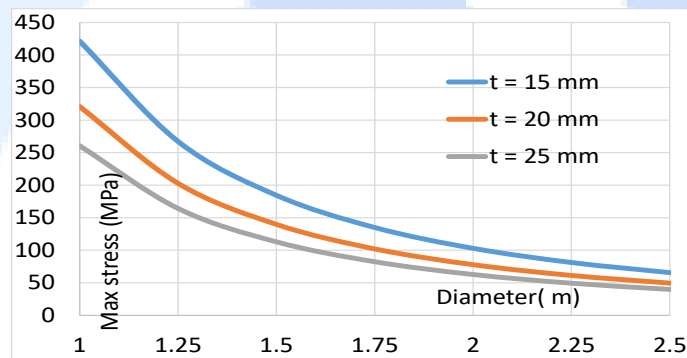


Figure 6. Max stress at base versus diameter and thickness

The main function of the cores being to resist wind load, 4 decommissioned shafts can be used as stiff cores. The width of the building equals 32 m, its height reaching 17 m. The environment is in the area of basic wind speed = 26 m/s. Simulations have been made, assuming several values of the re-used shaft's diameter and plate thickness. According to EN 1991-1-4 [8] the basic wind pressure equals 1.368 kN/m^2 , including the gust effect and the force coefficient. The cores being identical, the bending moment at the base of the cylinders equals $1.368 * 32 * 0.5 * 17^2/2 = 3162.816 \text{ kNm}$.

The graph of figure 6 exhibits the stress at the columns base, including the partial safety factor of $\gamma = 1.5$, considering wind as the primary variable load. Clearly light shafts from small-size turbines with small thickness of 15 mm can be sufficient for this purpose. High-rise buildings would require heavy shafts, this probably being not an interesting application.

3.5. Footbridges

If a decommissioned turbine shaft is cut in two halves lengthwise, and both ends are connected, a tapered semi-circular half cylinder is obtained. When it is placed in horizontal position, a new structure is created, which is able to carry vertical and horizontal loads. The semi-circular element can be the load-carrying part of a footbridge. Figure 7 exhibits a view from above; the widest part of the retrieved tower being located at the ends. To the right, figure 7 displays a 3D-view of the footbridge end.

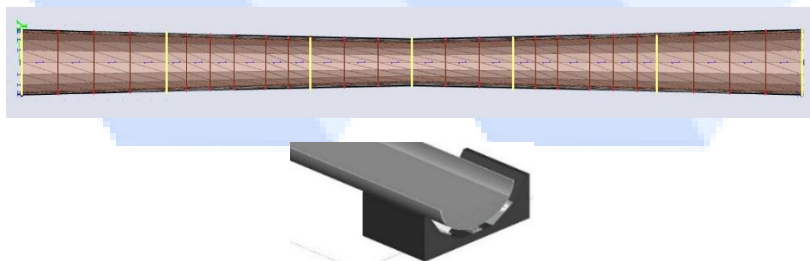


Figure 7. Footbridge from re-used wind turbine tower

Presently, footbridges are a promising application. For this reason, the research is concentrating on this item.

4. Requirements for Re-Use as Footbridges

In this section, the requirements for a footbridge, consisting of retrieved towers of wind turbine are listed. Some are generally valid and require no further explanation, but others are specific to footbridges made from wind turbine towers. The layout of the cross-section of the footbridge is exhibited in figure 8.

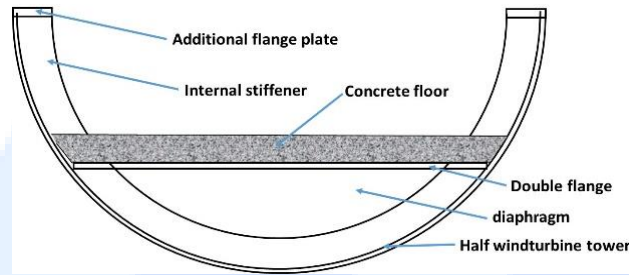


Figure 8. Principle of footbridge

The walkway is located in the lower half of the cross-section, the upper part of the semi-circle serving as parapets. If necessary, the internal stiffeners are supplemented by a diaphragm with double flange plate, supporting the concrete floor, with span equal to the stiffener distance. The latter can be reversed, without additional diaphragm, whereby the concrete walkway must bear the load in a transverse direction. Since the upper edge of the cross-section is compressed, this area is prone to buckling and can be supplemented by a horizontal flange plate, as shown in figure 8. The concrete floor is close to the horizontal gravity line of the cross-section area. Therefore, its contribution to the resisting moment is rather low and can be neglected.

4.1. General Requirements

Apart from the fabrication and hoisting and assembly, the design of footbridges has only few requirements to satisfy. The structure has to resist vertical loads due to the dead weight, the weight of the concrete floor and, if present, a thin pavement layer. The vertical traffic load complies with EN 1991-2 [9] and is a distributed load. In addition, horizontal loads accompany the traffic load.

An important effect is the wind load, as defined in EN 1991-1-4 [8]. Yet, the structure will certainly be slender and this implies resonance and vortex may occur. To mitigate these effects, the code requires the vertical natural frequency to exceed 1.25 Hz and its horizontal counterpart to be larger than 0.5 Hz. In practice, these limits are difficult to ensure.

Finally, thermal effects may have important effect on the load-carrying capacity of footbridges. If the structure can freely expand or retract, a general temperature rises or cooling does not affect the structure. However, a temperature difference between exposed parts and the areas in the shade can exist and introduce thermal stress.

4.2. Particular Requirements

As mentioned already, wind vortices are of concern for slender structures. The phenomenon can be analyzed by verifying the Strouhal and Scruton numbers. For this, the effective modal mass and the fundamental frequencies must be known. The Strouhal number is determined by the fundamental frequency multiplied by the width of the footbridge and divided by the average wind speed. The actual criterion of vortex shedding is found from the Scruton number as given by equation (3).

$$S_c = \frac{2 \delta_s m_e}{\rho b^2} \quad (3)$$

In equation (3) δ_s denotes the logarithmic damping decrement (steel = 0.01), m_e denotes the effective mass, ρ equals the air density (1.25 kg/m³) and b the structure's width. If S_c does not exceed 20, vibration by vortices will not be induced. This type of analysis supersedes any limitation of natural frequencies, which are recommended in the codes.

The initiation of human induced vibrations applies in particular to footbridges. This phenomenon can accurately be analyzed through the use of [9], which recommends a set of equations. In addition, the reference allows to distinguish between footbridges carrying 4 types of persons traffic. The method also considers the lock-in phenomenon, by which a number of pedestrians can make the damping effect of the bridge vanish and leads to sudden increase of vibrations.

Finally, the most characteristic requirement for structural steel parts, which have been exposed to nature, is the fatigue damage these parts may already have acquired. During the life time of the turbine tower, it has been exposed to the effect of wind gusts and of the rotation of the blades. The frequency of wind gusts depends on the location. At the North Sea coast these gusts appear 4 times a year, during 24 h with a period of 11.2 seconds. Hence, during the operation time the total number of cycles would be given by equation (4).

$$n_1 = \frac{25 \text{ ys } 4 \text{ days } 24 \text{ hrs } 3600 \text{ s}}{11.2 \text{ s}} = 771429 \text{ cycles} \quad (4)$$

These wind gusts introduce a stress variation denoted as $\Delta\sigma_1$. During the wind gusts, the effect on the tower is large and the turbine will be out of use. The effects of the rotor rotation introduce a larger number of cycles and is generated by the torque of the rotor. If the turbine works effectively during 30% of the time and it rotates at 20 RPM, the number of cycles is found from equation (5).

$$n_2 = 0.3 * 25 \text{ ys } 365 \text{ days } 24 \text{ hrs } 60 \text{ min } 20 \text{ rounds} = 78\,840\,000 \text{ cycles} \quad (5)$$

These cycles produce a stress variation of $\Delta\sigma_2$. As the probability of both effects resulting at the same location is evident, both may be added, although this may be a conservative approach. The largest values of $\Delta\sigma_1$ and $\Delta\sigma_2$ is found at the base of the tower and equal M/W , M denoting the bending moment and W the resisting moment. If w_{eff} denotes the effective wind pressure on the blades and tower, h denotes the tower height P the total power delivered by the turbine, b the tower diameter and ω the rotations per minute, A_{blades} denoting the area of the rotor blades, the bending moment at the base of the tower, due to wind equals the value in equation (6).

$$M_{wind} = \frac{w_{eff} b h^2}{2} + w_{eff} A_{blades} h \quad (6)$$

The bending moment at the tower base, due to the torque is found from equation (7).

$$M_{torque} = \frac{P}{\omega} \quad (7)$$

4.3. Case of 53.6 m High Wind Turbine

To obtain a realistic idea about the re-use of decommissioned wind turbine towers as footbridges, the case of a 53.6 m high shaft with 1.6 m diameter and plate thickness of 25 mm has been analysed. The effective wind pressure was calculated as 0.83 kN/m².

Fatigue verification

During its working time as part of a wind turbine, the tower base has experienced the stress variation by wind gusts, found from equation (6).

$$M_{wind} = 0.83 * 3 * 39 * 53.6 + 0.83 * (3.5 + 1.75 * 0.5 * 53.6^2 / 2) = 4592 \text{ kNm}$$

If the power of the turbine equals 1.5 MW and the rotation period equals 20 RPM, the torque is found from equation (7).

$$M_{torque} = 1500 \text{ kW} / 20 \text{ RPM} = 1500000 \text{ Nm} * 60 / 20 = 4500 \text{ kNm}$$

The resulting stresses are $\Delta\sigma_1 = 10.32 \text{ MPa}$ and $\Delta\sigma_2 = 10.11 \text{ MPa}$

If the fatigue class of the structure equals $\Delta\sigma_c = 80 \text{ MPa}$, and adopting Miner's rule, the damage at 25 ys of age is found from:

$$D = \frac{78\,840\,000}{\left(\frac{58.94}{10.11}\right)^5 \left(\frac{80}{58.94}\right)^3 2 \cdot 10^6} + \frac{771429}{\left(\frac{80}{10.32}\right)^3 2 \cdot 10^6} = 0.0023 + 0.0008 = 0.0031$$

However, EN1993-1-9 states a partial safety factor of 1.35 should be included. This implies all stress variations should be increased by this factor. The Damage then increases to $0.0105 + 0.0020 = 0.0125$. Hence, another 0.9875 damage may occur during the life of the tower as footbridge. Re-use of the 53.6 m tower into a footbridge of the same span was considered. To prevent local buckling due to compression, the number of diaphragm stiffeners of the original tower has to be increased, in order to comply with EN 1993-1-5 [10]. To obtain a factor $\rho=1$ as the multiplier of stresses, the distance of diaphragms is reduced to 0.60 m. This is a conservative approach, since the method of the standard is based on flat plates, whereas curved plates have a distinctly higher buckling load [11].

Human induced vibrations require higher stiffness than the result relating to the fundamental lateral mode of 1.01 Hz frequency and vertical mode of 2.18 Hz exhibited in figure 9.

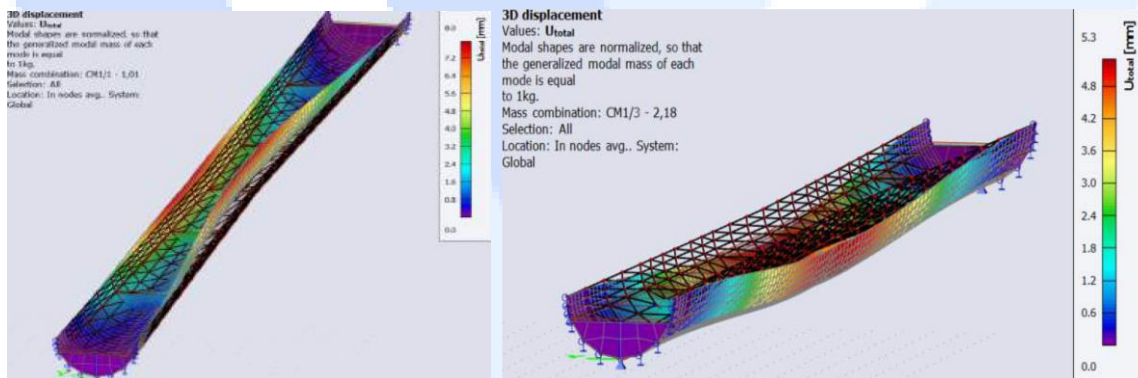


Figure 9. Free vibration modes of 54 m footbridge

However, more refined calculation results in maximum accelerations of the footbridge of 0.26 m/s², which is below the limit of 0.30 m/s² for comfort class 2. In addition, the critical wind speed equals 29.5 m/s and largely exceeds the threshold of 1.25-times the mean wind speed or 24.2 m/s.

A systematic parameter study was performed, allowing to increase insight in the results and possibly to obtain an approximation for re-use of wind turbine towers as simply supported footbridges. The graph of figure 10 exhibits 5 curves for corresponding span of the footbridges. This graph corresponds to limiting ULS-stress to 355 MPa and may be of practical interest.

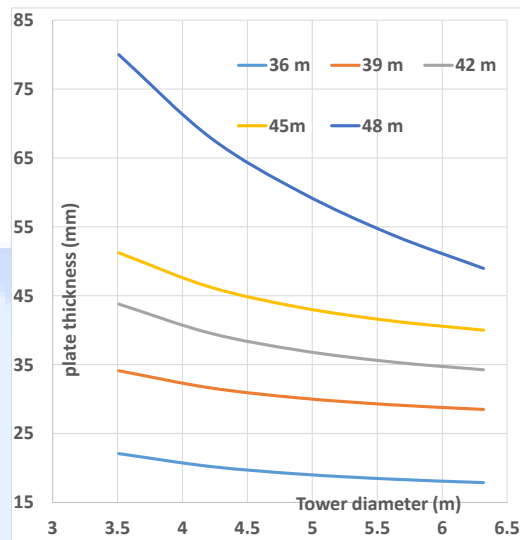


Figure 10. Diameter versus plate thickness for various values of footbridge span

Nomenclature

All symbols used are explained in the text.

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Presentation of a Proposed Model of the Argos Hellinikon Pyramid, Based on its Original Literature found Dimensions and on its Newly Measured Slopes, using a Specially Contrived T-FORM GEOMETRY TOOL

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Keywords

*Argos Hellinikon,
Pyramidal slopes,
Geometry,
Clinometer
Clissimeter
Argos, Pausanias,
Golden root symmetries,
American School of Classics
Athens.*

Abstract

This Paper is dedicated to my Personal Endeavour to Measure (by contriving a T-Form Geometry Tool, a Clissimeter/Clinometer) as far Accurate as Possible the Slopes of the Ancient Pyramid located in the Hellinikon Argos, near to Sparta and Mycenae, the places from where the War of Troy (started the 12BC century according to Homer's Iliad and Odyssey-ancient location Hisarlik).The reason of this interest of mine , was THAT the literature information concerning its slopes gave the value of 60 degrees! For my knowledge from the Pharaonic Pyramids in Egypt, a single slopes' value is associated with a SQUARE BASE Pyramid. Here the Argos Pyramid has an Orthogonal Parallelogram Plan View Plot, with a Particularity of an Entrance at one of its corners, making its 4 Sides' Structures somehow more complex to analyze I anticipate, by its Designers. Further the Structure is INCOMPLETE and it is extensively Damaged. The Pyramid was analyzed by the American School of Classics, Athens in the past. The Pyramid was considered serving as a FRYKTORIA [for light -Smoke Signals' Transitions – even used for Troy's Fall Messaging News]. Pausanias a Traveler and Geographer of the 2nd Century referred to this Pyramid.

1. Introduction

The main part of this paper involves the contrivance of a specially devised tool for measuring the slopes of the Argos Pyramid, which is a small sized ancient Colouros unfinished geometric structure, used as a fire/smoke signals transmitting beacon – a Fryktoria (Φρυκτωρία). With reference to Wikipedia: Greek pyramids, also known as the Pyramids of Argolis, refers to several ancient structures located in the plains of Argolid, Greece. The best known of these is known as the Pyramid of Hellinikon (Greek: Πυραμίδα του Ελληνικού). During the time of the geographer Pausanias, it was considered to be a tomb. Twentieth century researchers have suggested other possible uses. The surrounding country of Apobathmi was termed Pyramia (Πυράμια), from the monuments in the form of pyramids found there. The pyramid in Hellenikon was excavated first circa 1900 by Theodor Wiegand, who essentially removed all the fill from the floor. Later, in 1937, more excavation was made by the American School of Archaeology at Athens by the direction of L. Lord who concluded that both the structure at Ligurio and the one at Cephalaria were "guard houses capable of accommodating a small garrison who could control the countryside and be safe behind their walls from surprise attacks by a few persons». The Pyramid is located on an ancient road which once led from Argos to Tegea. It is preserved in good condition and it consists of a unique phenomenon and an archaeological mystery. It is situated nine kilometers away from Argos, near the springs of the Erasinios river on the main road which in antiquity led from Argos to Tegea. The pyramid is a live proof of mythology suggesting that there was a relation between the land of Argos and the Egyptian civilization. ACCORDING TO WIKIPAEDIA: "...The dimensions of the rectangular building surrounding the pyramid of Hellinikon are 7.03 meters by 9.07 metres. The external walls rise at a 60-degree angle up to 3.5 metres. The walls then become vertical in order to support the floor of the building. The entire monument is built from grey limestone from the district in the form of large blocks used in a trapezoidal/partially polygonal system...." [A22].



Figure 1. Argos Hellinikon Pyramid *Photo by PANAGIOTIS STEFANIDES 10 JULY 2025 -Monument dates [A20] to Mycenae Era (1600BC – 1100BC)

2. Pyramid Structure Configuration



Figure 2. Photos From Literature [A20].

Publications state that the slope of the faces of this pyramid is 60 degrees. The pyramid has a rectangular parallelogram base, I decided that I should take some measurements myself, knowing that a pyramid it was anticipated that this type of structure should have at least two slopes. A Tool was CONTRIVED (of a T-FORM GEOMETRY) for Measuring the SLOPES anticipating the difficulties involved in this process, despite the difficulties of the ROUGH AND UNEVEN SURFACES OF THE VOLUMETRIC STRUCTURES (with conditions of very high temperatures, July 10 and 19, 2025). No slope was detected at 60 degrees. The slope of the eastern front was measured to be 65 degrees, and the other three 70 degrees each. Basing this information on the calculations, a model of the pyramid was constructed with an ECCENTRICITY (approx. 1.06 m) Invented for the point of the vertical height that it meets the base of the Pyramid (UNIQUE FEATURE OF THIS PYRAMID MODEL, THAT THE CENTER OF ITS PLOT DIAGONALS' CROSSING POINT IS NOT THE SAME AS THE BASE CENTER CROSSING POINT OF THE VERTICAL HEIGHT -THERE IS ALSO A PECULIARITY OF THE PYRAMID'S INVOLVING AN ANGULAR ENTRANCE UNIQUE STRUCTURE, FOR A PYRAMIDS FORM). SLOPES FOUND OF 70 DEGREES AND 65 APPROXIMATE VALUES By Panagiotis Stefanides, ANNULLES THE 60 DEGREES SLOPES STATED BY CONCERNED LITERATURE. A PARALLELOGRAMME ORTHOGONAL PYRAMID CANNOT HAVE A SINGLE VALUE SLOPE FOR ITS ALL 4 FACES (AS WITH THE GREAT PYRAMID EXAMPLE OF SQUARE BASE – GIZA CAIRO PYRAMID'S SLOPE 51 deg 49 first 48 seconds).

3. Pyramid Historic References

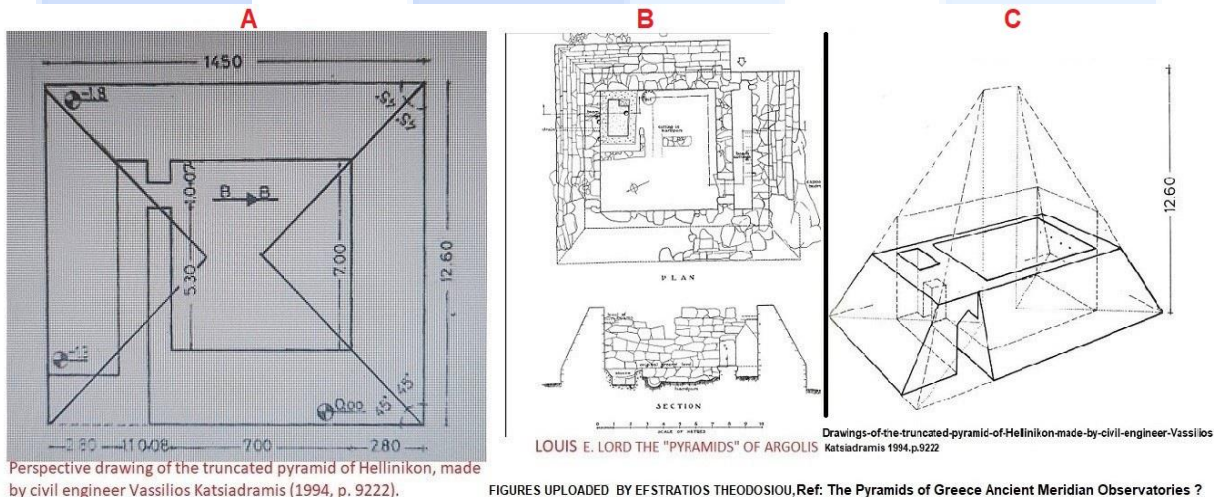


Figure 3. Figures Uploaded by EFSTRATIOS THEODOSIOU [A1]. Photo A and B ref: LOUIS E. LORD American School of Archaeology at Athens [A2]. The "Pyramids" of Argolis

Figure 3, Photo C: A Model of the Pyramid found in literature, (by Civil Engineer Vassilios Katsidramis) having 2 top Height Edges joined together. PHOTOS A and B From the Louis E. Lord report: THE "PYRAMIDS" OF ARGOLIS. A report of a brief investigation structures in Argolis of by The Archaeological Institute of America, conducted under the auspices of The American School of Classical Studies at Athens, August 2-11, 1937.

Following photo is the PLAN VIEW OF THE PLOT of the Pyramid in Hellenikon. Ground PLOT: excavated first circa 1900 by Theodor Wiegand (Figure4.), who essentially removed all the fill from the floor.

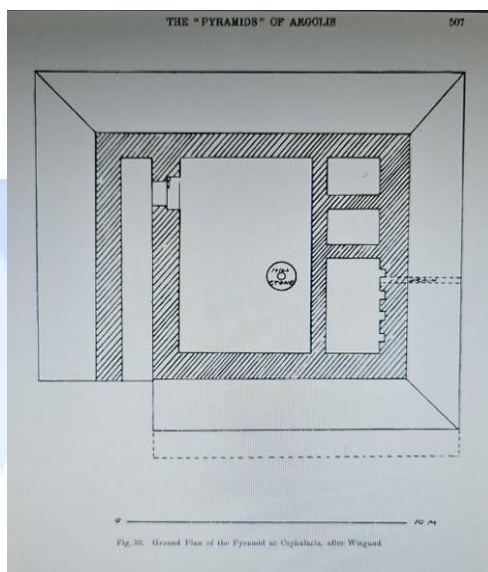


Figure 4. PLAN VIEW OF THE PYRAMID BASE PLOT By Theodor Wiegand - Photo Reference by LOUIS E. LORD American School of Archaeology at Athens: The "Pyramids" of Argolis. The PYRAMID in KEPHALARI HELLINIKON was excavated first circa 1900 by THEODOR WIEGAND, who essentially removed all the fill from the floor. Later, in 1937, more excavation was made by the American School of Archaeology at Athens.

4. Slopes' Measurements via The Contrived T-FORM GEOMETRY Device (CLINOMETER /CLISSIOMETER by Panagiotis Stefanides)

The tool is of T-FORM CONFIGURATION -CALIBRATED By Panagiotis Stefanides to directory indicate the slope of the Pyramid Face , while colouring by various colours the number for recognition of it from a relevant distance particularly for the slope of the WEST WALL , presenting a real difficulty for its slope measurement as Figure 5 (left side) indicates (operation by sliding the top horizontal number of degree obtained, visually , keeping the vertical part of the tool steadily orthogonal , as far as possible). The calibration of the TOOL is shown on the right-hand photo. NOTE: the horizontal rod passes via a hole on the top of the vertical rod and is capable of reciprocation "IN_OUT"!

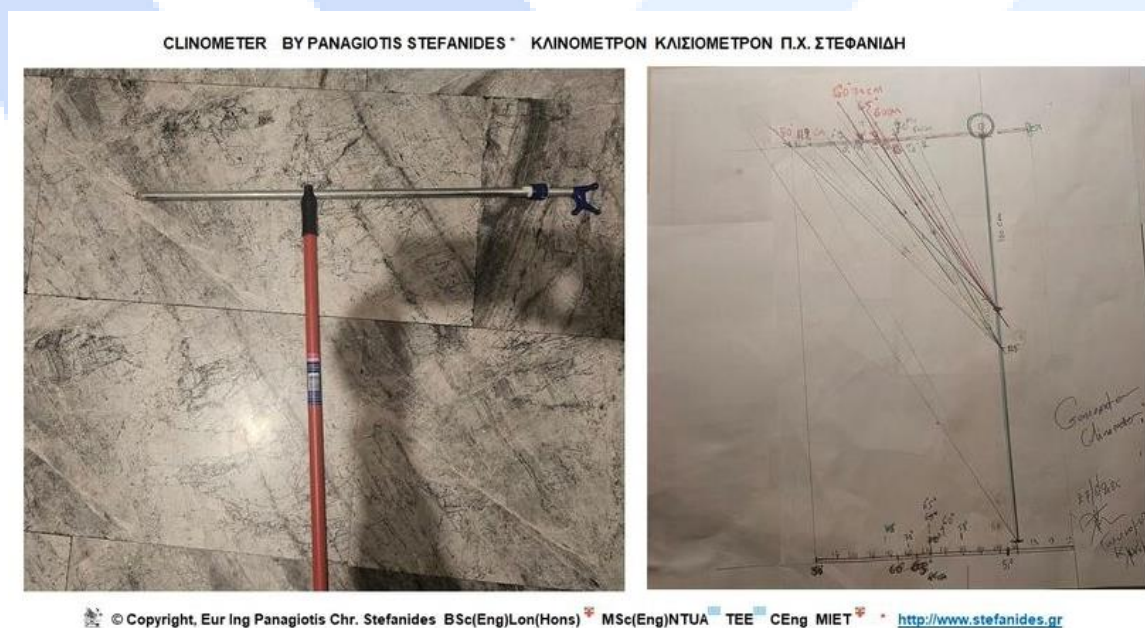


Figure 5. T-FORM GEOMETRY TOOL- CLINOMETER /CLISSIOMETER by Panagiotis Stefanides (Structure and Calibrating Measuring Rod Photos)

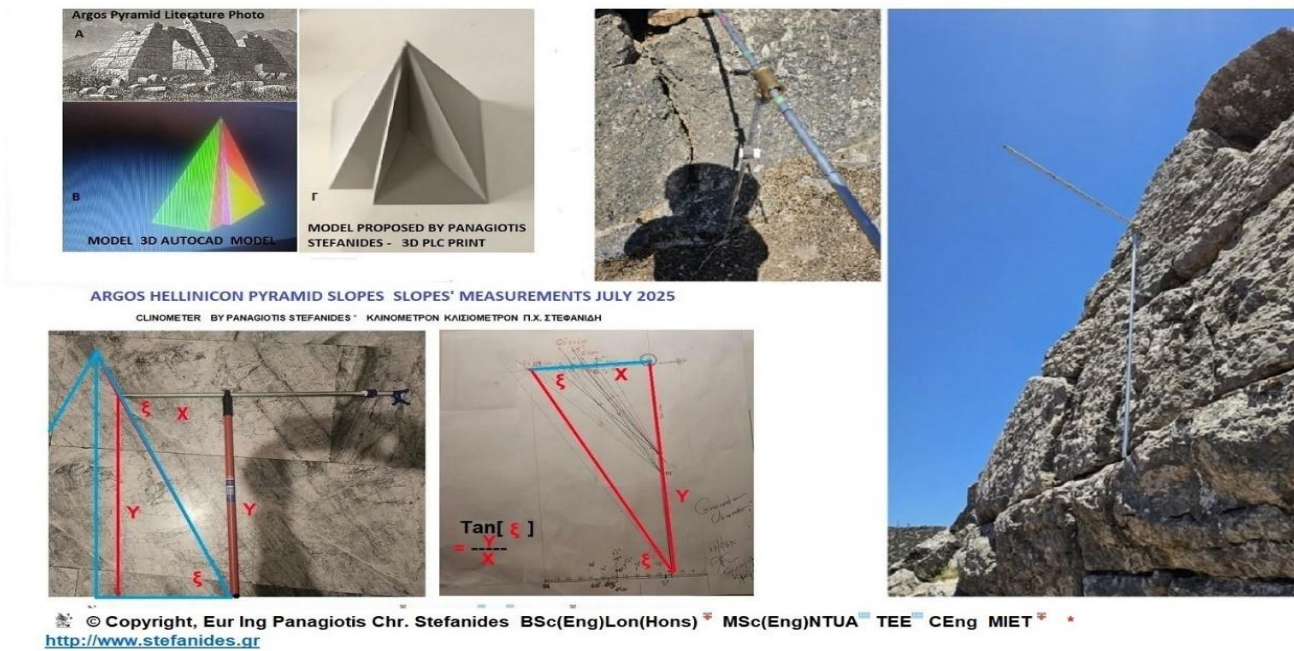


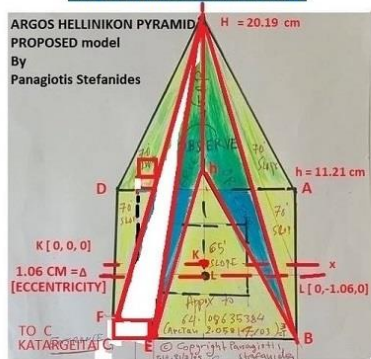
Figure 6. T-FORM TOOL MEASUREMENTS. Right Hand Picture MEASUREMENTS ON THE WALL OF THE SOUTH WALL OF THE PYRAMID. MEASUREMENTS' reading relied SOLELY on VIEWING from distance THE COLOUR of the SLOPE "angle number"



Figure 7. Measurements by the T-FORM GEOMETRY TOOL. The COLOURED Regions of the SLOPES' NUMBER shown

5. Proposed Model by Panagiotis Stefanides

According to the following work of designing a model of the Pyramid, based on the relevant applicable geometries and vectors as specified by the Author Panagiotis Stefanides and by the relevant design information of the proposed model relayed for a 3D AUTOCAD MODEL to be developed by, colleague, Engineer Dr. GIANNIS KANDYLAS, which was further Printed (Figure 8. B) to a 3D PLC form by Engineer A. GEORGOSTATHIS (Figure 8. Γ).



ΚΑΙΝΟΥΡΓΙΟ:
 ΤΟ C ΚΑΤΑΓΕΙΤΑΙ.

ΓΡΑΦΟΥΜΕ ΤΑ ΤΡΙΓΩΝΑ:
 EFH
 EhH

HhB

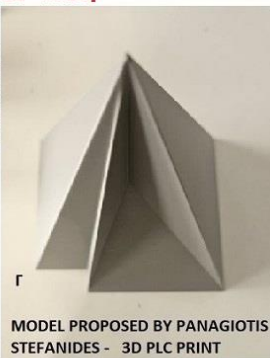
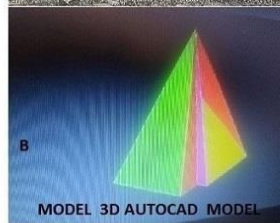
ΟΙ ΣΥΝΤΕΤΑΓΜΕΝΕΣ
 ΙΔΙΕΣ ΟΠΩΣ ΕΔΟΘΗΣΑΝ
 ΠΡΟΗΓΟΥΜΕΝΩΣ

• FCh:
 F(-7.35,-4.51,0)-
 C(-7.35,-6.29,0)-
 h(0,-1.06,11.79)
 • CEh:
 C(-7.35,-6.29,0)-
 E(-3.25,-6.29,0)-
 h(0,-1.06,11.79)
 • EBh:
 E(-3.25,-6.29,0)-
 B(7.35,-6.29,0)-
 h(0,-1.06,11.79)
 • Bhh:
 B(7.35,-6.29,0)-
 h(0,-1.06,11.79)-
 H(0,-1.06,20.19)

BAH: B(7.35,-6.29,0)-
 A(7.35,6.29,0)-
 H(0,-1.06,20.19)
 • AGH: A(7.35,6.29,
 0)-G(-3.25,6.26,0)-
 H(0,-1.06,20.19)
 • GDH:
 G(-3.25,6.26,0)-
 D(-7.35,6.29,0)-
 H(0,-1.06,20.19)
 • DFH:
 D(-7.35,6.29,0)-
 F(-7.35,-4.51,0)-
 H(0,-1.06,20.19)
 • FHH:
 F(-7.35,-4.51,0)-
 h(0,-1.06,11.79)-
 H(0,-1.06,20.19)

Monument dates to Mycenaean Era [1600 BC - 1100BC]

Argos Pyramid Literature Photo



A. LITERATURE PHOTO OF ARGOS PYRAMID.

B. AUTOCAD BY GIANNIS KA DYLAS.

Γ. 3D PLC PRINT BY 3D FORM - AP. GEORGOSTATHIS

GEOMETRY AND VECTOR DRFINITION BY PANAGIOTIS STEFANIDES

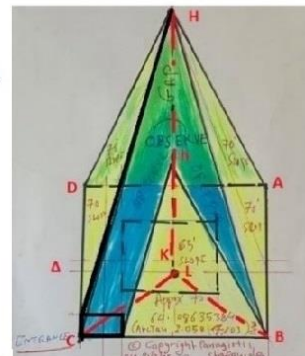


Figure 8. COORDINATES' employment on a BASE DESIGNED according to the PLAN BASE VIEW of the PLOT SCALING by LOUIS E. LORDS Literature [A2]

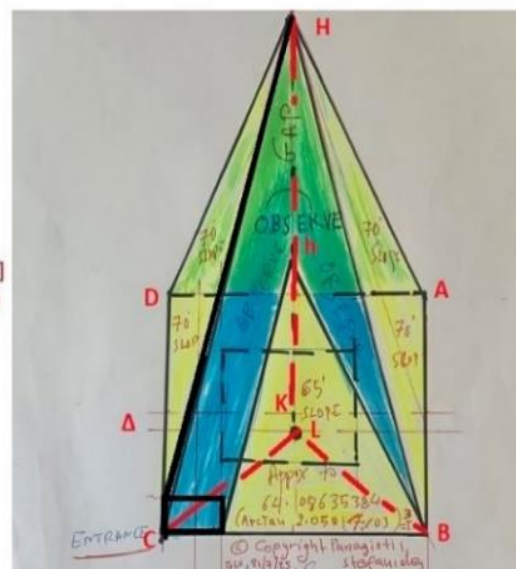
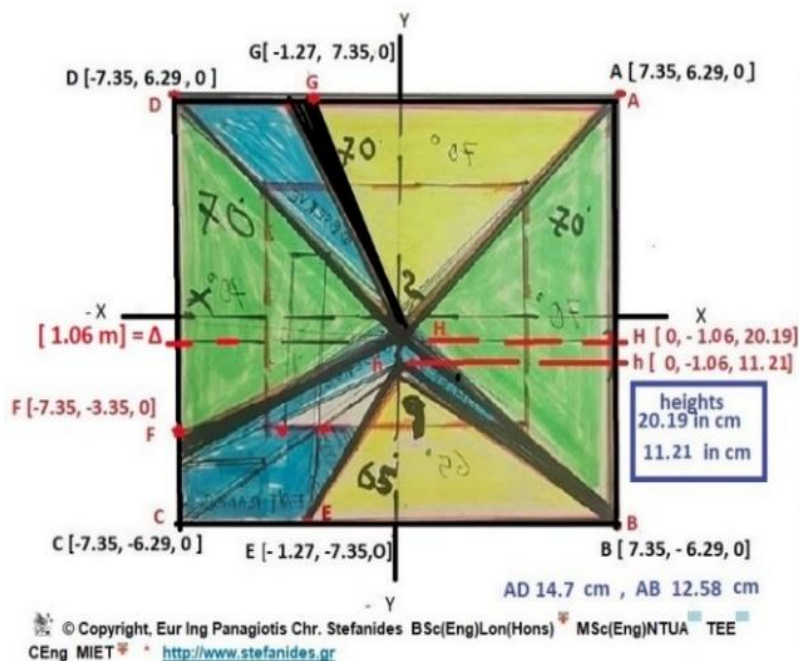


Figure 9. Design of the Proposed Pyramid Model

The slope of the Eastern Front was measured to be 65 degrees, and the other three 70 degrees each. Introducing this information on the calculations, a model of the pyramid was constructed with an eccentricity invention of the point of the vertical height at the base of the Pyramid (A UNIQUE FEATURE OF THE PYRAMID MODEL THE CENTER OF THE INTERSECTION OF THE DIAGONALS OF THE BASE IS NOT THE SAME AS THE CENTER OF THE VERTICAL HEIGHT -THERE IS ALSO AN INTERNAL ANGLE OF ENTRY INTO THE PYRAM). SLOPES 70 DEGREES AND 65 APPROXIMATE VALUES By Panagiotis Stefanides. THIS ANNULLES THE 60 DEGREES SLOPES STATED BY CONCERNED LITERATURE, FROM STUDIES CIRCA 1938! A PARALLELOGRAMME ORTHOGONAL PYRAMID CANNOT HAVE A SINGLE VALUE SLOPE FOR ITS ALL 4 FACES .AS WITH THE GREAT PYRAMID EXAMPLE OF SQUARE BASE (GIZA-Slope 51 deg 49 first 48 seconds). There has been introduced an ECCENTRICITY of approx. 1.06 m on the - X axis for the position of the HEIGHT AND VERTEX. (As base Orthogonal Parallelogramme- and not a Square). It is anticipated that slopes and base plot dimensions are related to various forms the SQUARE ROOT OF THE GOLDEN NUMBER! (70 deg approx. to 69.09484255 and the 65deg. approx. to 64.08635384 deg.). Based on the above research performed, a Theoretical Model of the Pyramid was designed Computed in a Software AutoCAD form, based on the Geometry of its Shape, Space, Slopes, and the Vectors of its Geometry Involved and, a 3D PLC MODEL PRINTED.



Figure 10. FORM TOOL GEOMETRY CONTRIVANCE by Panagiotis Stefanides. Argos Pyramid's SLOPES MEASUREMENTS 10 JULY 2025

6. Discrete Geometry Golden Root Slopes' Relationships Proposed

It is anticipated that slopes and base plot dimensions are related to various forms the SQUARE ROOT OF THE GOLDEN NUMBER! Based on original information, the work appearing in Figure 11 (based solely on photos and literature) was developed dating (June 2025). Here it is anticipated difference of angles involved estimated, just, from photography Figure 11. INTERESTING HERE IS THE GOLDEN ROOT RELATIONSHIPS APPROACHED FURTHER AND FOR THE ANGLE OF 70 DEGREES FROM THE IN-SITU MEASUREMENTS ANTICIPATED THAT APPROXIMATE VALUE OF 65 DEGREES ANGLE IS 64.08635384 WHOSE TANGENT EQUALS 2.618033989 WHOSE GOLDEN ROOT RELATED FORM IS: THE CUBE OF T, WHERE T IS THE SQUARE ROOT OF THE GOLDEN NUMBER ($\Phi = 1.618033989$): 1.27201965. Similarly: ANTICIPATED THAT APPROXIMATE VALUE OF 70 DEGREES ANGLE IS 69.09484255 WHOSE TANGENT EQUALS 2.05817103 AND WHOSE GOLDEN ROOT RELATED FORM IS: THE FOURTH POWER OF T (2.618033989 = Φ SQUARED), WHERE T IS THE SQUARE ROOT OF THE GOLDEN NUMBER ($\Phi = 1.618033989$): 1.27201965. AN APPROXIMATE AREA OF THE PLAN VIEW PLOT OF THE PYRAMID IS: 15X11.727066 SQUARE MWETERS, RELATED ANGLE OF THE SIDES RATIO BEING 1.27201965 WHOSE ARCTAN = 51.82729239 (CAIRO GIZA GREAT PYRAMID'S SLOPE).

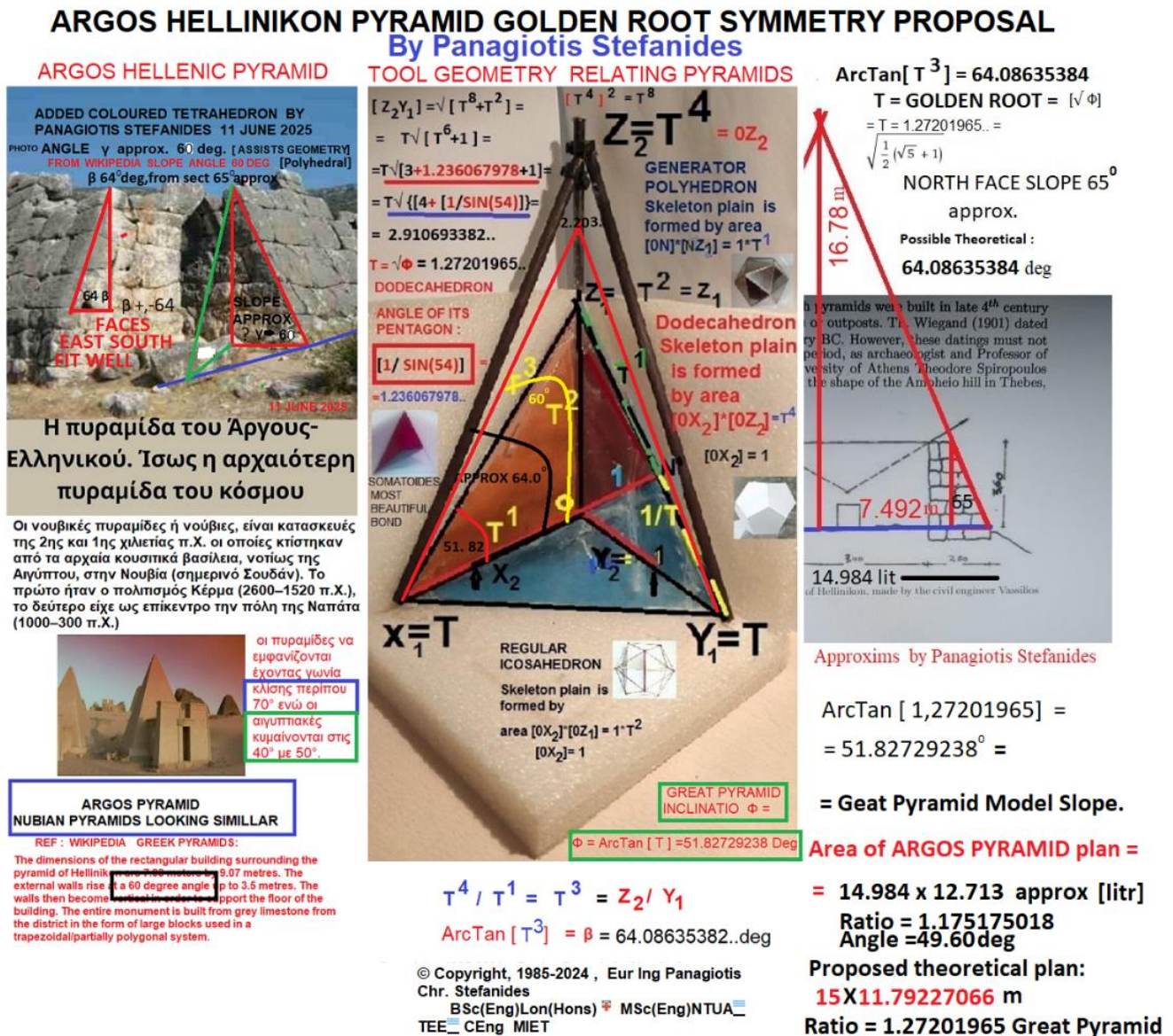


Figure 11. Work based solely on photos and literature. Golden Root Relationships of the pyramid geometry PROPOSED by AUTHOR P.Stefanides

7. GOLDEN ROOT DISCRETE GEOMETRY based on PLATO'S "MOST BEAUTIFUL TRIANGLE" Proposed by Panagiotis Stefanides "DIACHRONICALLY" to National and International CENFERENCES, SYMPOSIA AND EXHIBITIONS [3, 4, 5]

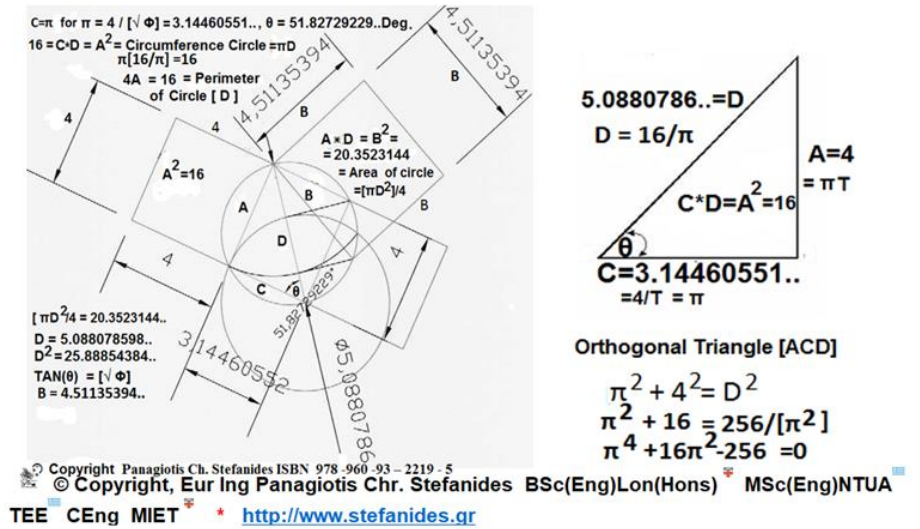
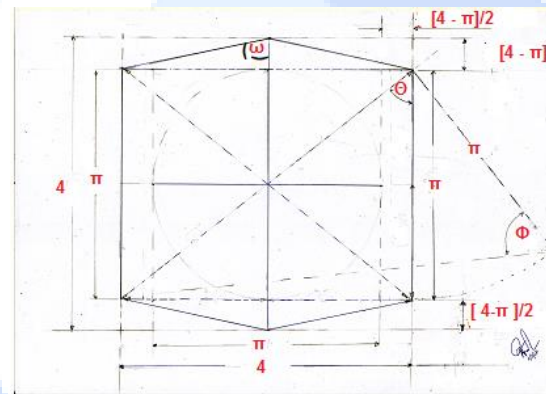


Figure 12. GOLDEN ROOT DISCRETE GEOMETRY Based Interpretation of PLATO'S TIMAEUS "MOST BEAUTIFUL TRIANGLE" BY PANAGIOTIS STEFANIDES [A21], Diachronically Disseminated Nationally and Internationally



INSCRIBED TO SQUARE OF PERIMETER 4π ,
 CIRCLE WITH CIRCUMFERENCE π^2
 $\text{TAN}[\theta] = 4/\pi$, [for $\pi = 4/T$], $T = \text{SQRT}\{[(\text{SQRT}(5) + 1)/2]\}$
 $\text{TAN}[\Phi] = \text{SQRT}[4^2 + \pi^2]/\pi$, [for $\pi = 4/T$]
 $T = 1.27201965\dots$, $\theta = 51.82729237\dots$ deg.
 $\pi = 3.14460551\dots$, $\Phi = 58.28252559\dots$ deg.
 $[4 - \pi]/2 = 0.427697244\dots$ for $\pi = 4/T$
 $\text{TAN}[\omega] = (4/2) / [4 - \pi]/2 = 4/[4 - \pi] = 4.676205016\dots$
 $\omega = 77.92918912\dots$ deg., $90 - \omega = 12.07081088\dots$ deg.

Figure 13. GOLDEN ROOT DISCRETE GEOMETRY Basic Generating DISCRETE POLYHEDRAL Forms Based on PLATO'S TIMAEUS "MOST BEAUTIFUL TRIANGLE" [3] as Interpreted and Proposed by Panagiotis Stefanides

8. Conclusions

The presentation of this paper exposed the work undertaken by the AUTHOR RESEARCHER OF THIS CASE, Single Handling the Searching of the INCLINATION SLOPES of the ARGOS KEPHALARI HELLINIKON ANCIENT PYRAMID, as HE Imagined Conceptually THE EXISTING DEVIATION of Values, from the LITERATURE declared of a SINGLE ONE OF 60 DEGREES, Inappropriate Concept for a PARALLELOGRAMME BASE PYRAMID. It is proposed that in this modern ERA of Knowledge, Experience and Scientific Evolution that, the European, UNESCO and World Community should Rethink "again" of RE-EVALUATE this Tremendous Ancient Monument. At For the slopes' accurate measurements the easiest should be for a team to employ use of Walls' AREAS INCLINATION, by use of 3D latest Laser Measurement Methods.

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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- [10] (PDF) PYRAMIDAL SLOPE MEASUREMENT SIMPLE DEVICE CLIMOMETER BY PANAGIOTIS STEFANIDES ΚΛΙΝΟΜΕΤΡΟΝ ΚΛΙΣΙΟΜΕΤΡΟΝ ΠΑΝΑΓΙΩΤΗ Χ ΣΤΕΦΑΝΙΔΗ
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- [28] Greek pyramids - Wikipedia GREEK PYRAMIDS



Marshall Mix Design and Performance Evaluation of Hot Mix Asphalt Wearing Course BZH22

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Air voids,

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Abstract

This study evaluates a BZH22 hot mix asphalt wearing course using the Marshall mix design method, with emphasis on the effect of bitumen content on mechanical and volumetric performance. Six crushed-stone aggregate fractions were combined to obtain a dense gradation within the specified limits. Three mixtures containing 5.0%, 5.5%, and 6.0% bitumen by total mix weight were prepared, and Marshall specimens were compacted with 50 blows per face. Bulk density, air void content, Marshall stability, flow, and stiffness quotient were determined. The 5.0% mixture reached the highest stability, 8.535 kN, while stability decreased to 7.010 kN and 6.810 kN at 5.5% and 6.0% bitumen, respectively. Bulk density increased from 2.438 to 2.456 g/cm³ as binder content increased, whereas air voids decreased from 5.60% to 4.20%. Among the tested alternatives, 5.0% bitumen provided the best overall balance of stability, deformation response, and volumetric properties. The stiffness quotient at this level remained slightly below the stated target, indicating that minor gradation or binder adjustments could further improve performance. The results support the practical use of the Marshall approach for optimizing dense-graded wearing-course mixtures produced with local crushed stone aggregates.

1. Introduction

Hot mix asphalt (HMA) is widely used for flexible pavement surface layers because it can provide a durable and relatively smooth wearing course under traffic and environmental exposure. Its performance depends strongly on the balance between aggregate gradation, binder content, and compaction. Too little bitumen can produce a brittle and permeable mixture, while an excessive amount can reduce aggregate interlock and increase the tendency toward bleeding or permanent deformation [1,3].

The Marshall method remains a practical mix-design procedure because it combines volumetric measurements with mechanical indicators such as stability and flow. Although the method is empirical, it is useful for comparing trial mixtures prepared with the same materials and for selecting a binder range that offers a balanced response [1,2].

The present work investigates a BZH22 wearing-course mixture produced with crushed stone aggregates. The study aimed to establish a combined gradation within the specified limits, prepare mixtures at three nearby bitumen contents, evaluate their volumetric and Marshall properties, and identify the most suitable binder content among the tested alternatives.

2. Materials and Methods

The aggregate system consisted of five crushed-stone fractions and mineral filler. The proportions were selected to obtain a dense combined gradation suitable for a wearing course. Penetration-grade bitumen was used as binder. An initial design estimate of approximately 4.73% bitumen by total mixture weight guided the selection of three trial contents: 5.0%, 5.5%, and 6.0%.

Table 1. Aggregate fraction dosages used for the BZH22 mix

Aggregate fraction	Dosage (%)
0/2 mm	36
2/4 mm	13
4/8 mm	15
8/11 mm	23
11/16 mm	8
Mineral filler	5

For specimen preparation, the aggregate fractions were weighed, heated, and mixed with hot bitumen until a uniform coating was obtained. The mixture was placed in preheated Marshall molds and compacted with 50 blows per face. Three specimens were prepared for each binder content, giving nine specimens in total. After cooling, the specimens were removed from the molds and their dimensions were checked.

Bulk density was determined by hydrostatic weighing. Before the Marshall test, specimens were conditioned in a 60°C water bath for at least 40 min and then loaded at 50.8 mm/min. The peak load was recorded as Marshall stability and the deformation at peak load as flow. Air voids and stiffness quotient were evaluated from the measured data.

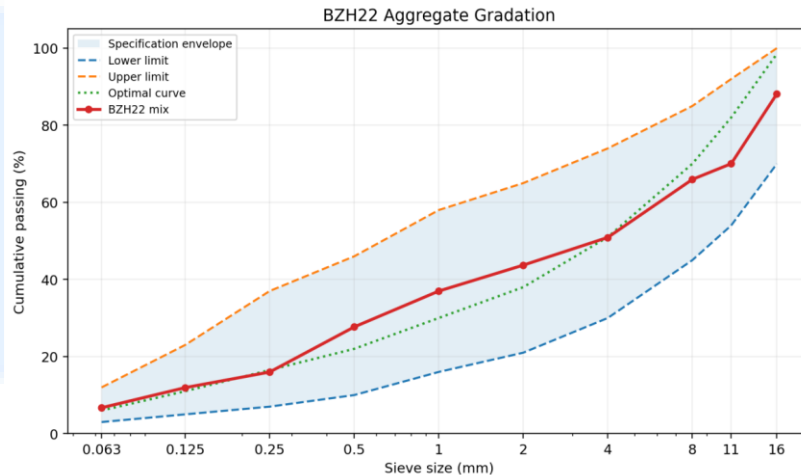


Figure 1. Combined BZH22 aggregate gradation relative to the specified lower and upper limits

3. Results and Discussion

The combined gradation remained within the specified limits over the full sieve range, confirming that the selected fractions produced a dense and continuous particle-size distribution. The curve was close to the central trend in the fine-to-intermediate range and became slightly coarser at the larger sieve sizes. This structure is consistent with the strong stability observed at the lower binder content.

Table 2. Marshall and volumetric results for the three bitumen contents

Bitumen (%)	Stability (kN)	Flow (mm)	Bulk density (g/cm ³)	Air voids (%)	Stiffness (kN/mm)
5.0	8.535	4.611	2.438	5.60	1.85
5.5	7.010	5.911	2.451	4.80	1.19
6.0	6.810	5.714	2.456	4.20	1.19

Marshall stability decreased as bitumen content increased. The 5.0% mixture reached 8.535 kN, exceeding the stated 8.0 kN minimum, while the 5.5% and 6.0% mixtures reached 7.010 kN and 6.810 kN, respectively. The trend suggests that additional binder progressively reduced the contribution of aggregate interlock to the load-carrying response. Similar behavior is commonly reported when binder content moves beyond the range that best supports the mineral skeleton [3,4].

Flow increased from 4.611 mm at 5.0% to 5.911 mm at 5.5%, followed by a small reduction to 5.714 mm at 6.0%. The higher deformation levels at the two richer binder contents are consistent with the reduction in stability. At the same time, bulk density increased slightly from 2.438 to 2.456 g/cm³, showing progressive filling of internal void space by the binder.

Air void content decreased from 5.60% to 4.80% and then to 4.20% as binder content increased. The 5.5% mixture fell clearly within the stated 4.5-5.5% target range, while the 5.0% value was marginally above the upper limit and the 6.0% value fell below the lower limit. The results illustrate the design trade-off between void filling and mechanical stability.

Considering all measured parameters, 5.0% bitumen was selected as the preferred content among the tested alternatives. It provided the highest stability and the most favorable overall balance of mechanical and volumetric performance. The stiffness quotient at this level was 1.85 kN/mm, below the stated 2.2 kN/mm target, so the mixture should not be described as satisfying every criterion without qualification. A modest adjustment of aggregate gradation or binder type could be investigated in future work to improve stiffness while preserving the stability advantage observed at 5.0% [4,5].

Marshall Mix Design Results - BZH22

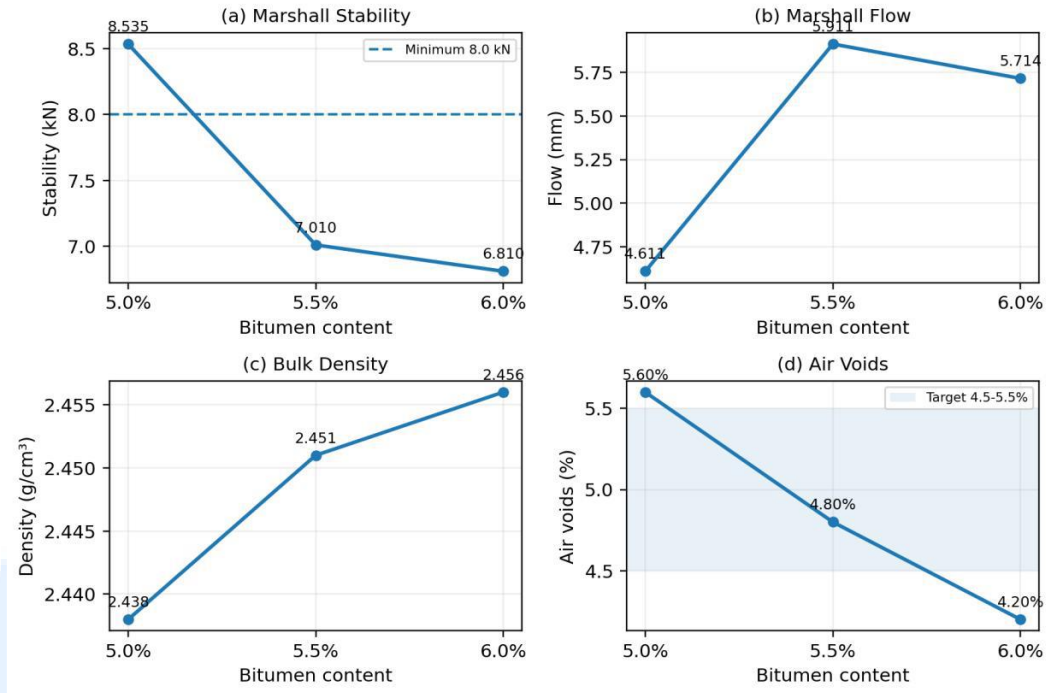


Figure 2. Summary of stability, flow, bulk density, and air void trends with bitumen content



Figure 3. Laboratory preparation and weighing of aggregate fractions for the Marshall specimens

4. Practical Implications

The laboratory results provide a practical reference point for BZH22 wearing-course production using the tested aggregate blend. In plant production, the selected proportions would still require normal corrections for aggregate moisture, temperature control, and production variability. Because relatively small changes in binder content produced clear changes in stability and air voids, quality control should focus closely on binder dosage, gradation, and compaction. Regular verification of Marshall properties and field density would be important before full-scale implementation.

5. Conclusions

- The selected aggregate fractions produced a dense BZH22 gradation that remained within the specified sieve limits.
- Marshall stability was highest at 5.0% bitumen (8.535 kN) and decreased at 5.5% (7.010 kN) and 6.0% (6.810 kN).
- Bulk density increased slightly with binder content, while air voids decreased from 5.60% to 4.20%.
- Among the three trial mixtures, 5.0% bitumen provided the best overall balance and was selected as the preferred design value.
- The stiffness quotient at 5.0% remained below the stated target, indicating that further refinement of gradation or binder properties may improve the mixture before broader field application.

Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Effect of Span Length and Damping on the Modal and Seismic Response of Three Long-Span Suspension Bridges

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Abstract

In this study, three long-span suspension bridges with substantially different main spans—the Bosphorus Bridge (1074 m), Osmangazi Bridge (1550 m), and 1915 Çanakkale Bridge (2023 m)—are comparatively investigated to evaluate the effects of span length and structural damping on their modal and seismic responses. Three-dimensional finite element models were developed in SAP2000 considering geometric nonlinearity, and their dynamic characteristics were verified against experimental/numerical reference values available in the literature. The fundamental frequencies decreased systematically with increasing span length, from 0.0790 Hz for the Bosphorus Bridge to 0.0510 Hz for the Osmangazi Bridge and 0.0425 Hz for the 1915 Çanakkale Bridge, while similar low-order modal patterns were retained. The three bridges were subsequently subjected to the same three-component Gebze record of the 1999 Kocaeli earthquake using damping ratios of 2% and 5%. Unlike the modal characteristics, the seismic response did not vary monotonically with span length; the Osmangazi Bridge exhibited the largest response for several deck- and tower-related quantities. Increasing the damping ratio from 2% to 5% reduced deck and tower responses by approximately 5–50%, whereas the total axial forces in the main cables and hangers changed by less than about 3%. The results demonstrate that span length has a systematic influence on modal characteristics, whereas seismic demand is governed by the combined interaction of structural properties, excitation characteristics, and damping.

1. Introduction

Long-span suspension bridges are critical infrastructure components that enable the continuous crossing of large water bodies while maintaining the continuity of national and regional transportation networks. Due to their low structural damping, high flexibility, complex cable-deck-tower interactions, pronounced geometric nonlinearity, and closely spaced vibration modes, these structures exhibit significantly more complex dynamic and seismic behavior than conventional bridge systems [1-3]. Türkiye, located within a highly seismic geography dominated by the North Anatolian Fault Zone (NAFZ), has become home to three of the world's longest-span suspension bridges over the past two decades: the Bosphorus Bridge, opened to service in 1973; the Osmangazi Bridge, opened in 2016; and the 1915 Çanakkale Bridge, opened in 2022, which holds the world record for the longest main-span suspension bridge [4].

Although located within a similar tectonic belt, these three bridges differ markedly in span length, deck geometry, tower height and cable systems. A systematic comparison of the modal and seismic behavior of structures exposed to the same regional seismic hazard yet differing in span length by roughly a factor of two is of considerable importance both for understanding the effect of span length on dynamic characteristics and for deriving engineering insights applicable to future long-span bridge design [5]. In addition to span length, the assumed structural damping ratio is a key analysis parameter that directly affects the amplitude of the dynamic response; therefore, evaluating the seismic performance of these bridges at different damping ratios (2% and 5%) provides further insight into how sensitive long-span suspension bridges are to this modeling assumption. The 2% damping ratio corresponds to the reference damping level adopted in the previously developed bridge models, while 5% was considered only as an additional comparative value for evaluating damping sensitivity. No retrofit or supplemental damping system was modeled, and the two damping ratios therefore represent alternative analytical assumptions rather than different physical configurations of the bridges.

The 1999 Kocaeli earthquake, which occurred in the Marmara Region and affected numerous provinces, produced one of the most important ground motion records used as a reference in the seismic design of engineering structures in the region. In this study, (i) geometric properties, (ii) finite element models and modal analysis results (natural frequencies and mode shapes), and (iii) structural responses under 1999 Kocaeli

earthquake records of three suspension bridges with different span lengths, located in the same region, are presented comparatively, together with an assessment of the effect of damping ratio (2% and 5%) on the obtained seismic responses. The originality of the study lies in the integrated comparison of three suspension bridges with different span lengths, located in the same seismic region, each modeled and separately validated, and evaluated under a consistent damping ratio framework.

2. Suspension Bridges

Suspension bridges are used to cross exceptionally long, unobstructed spans such as straits and bays, and their high flexibility, low structural damping, and cable-deck-tower interaction make them particularly sensitive to dynamic and seismic loading. The three bridges examined in this study illustrate this structural type across a wide range of span lengths, from 1000 m to about 2000 m, while sharing a common regional seismic setting.

The first, the Bosphorus Bridge, connects the Asian and European sides of Istanbul and has a main span of 1074 m. The bridge is distinguished from other suspension bridges by its aerodynamically shaped steel box-girder deck and its hangers, which were originally installed in an inclined (V-shaped) configuration; following damage to a hanger-deck connection near the European tower during a storm in January 2004, the inclined hanger configuration was replaced with a vertical hanger configuration in July 2015, and the data corresponding to this current (vertical hanger) configuration are used in the present study [5, 6]. The second, the Osmangazi Bridge (İzmit Bay Bridge), was constructed as part of the Kocaeli-İzmir (Otoyol 5) Motorway across İzmit Bay and has a main span of 1550 m, making it, at the time of its opening, the world's fifth-longest main-span suspension bridge; construction of its foundations began in 2013, and the bridge was opened to traffic in 2016, reducing the bay crossing time from approximately 2 hours to 6 minutes [7]. The third and longest, the 1915 Çanakkale Bridge, crosses the Dardanelles Strait between the districts of Gelibolu and Lapseki and has a main span of 2023 m, making it the world's longest main-span suspension bridge at the time of its completion; forming a critical section of the Kınalı-Tekirdağ-Çanakkale-Savaştepe Motorway, the bridge was opened to service in 2022, and its tower foundations, located approximately 20 km from the western branches of the North Anatolian Fault Zone, were constructed on freely seated, gravel-bed-supported caissons in deep-water conditions [8-10].

The main geometric and design parameters of the three bridges are comparatively presented in Table 1. As shown in Fig. 1, the main span length increases by approximately 88% from the Bosphorus Bridge to the 1915 Çanakkale Bridge. This increase is accompanied by substantial changes in deck width, tower height, main cable dimensions, and hanger arrangement, providing a broad geometric range for evaluating the influence of span length on the dynamic behavior of suspension bridges.

Table 1. Comparison of the geometric and design properties of the three bridges

Parameter	Bosphorus Bridge	Osmangazi Bridge	1915 Çanakkale Bridge
Year of opening	1973 (2015: hanger replacement)	2016	2022
Main span (m)	1074	1550	2023
Side span (m)	231 + 255	566 × 2	770 × 2
Total crossing length (m)	1560	2907	4608
Deck width (m)	33.40	35.93	45.06
Number of lanes	6 (3+3)	6 (3+3)	6 (3+3)
Tower height (m)	165	252	318
Main cable diameter	Ø58.0 cm	Ø78.1 cm	Ø86.9 cm
Number of hangers	236 (main)	210 (126 main + 42×2 side)	148 4 main + 32×2 side)

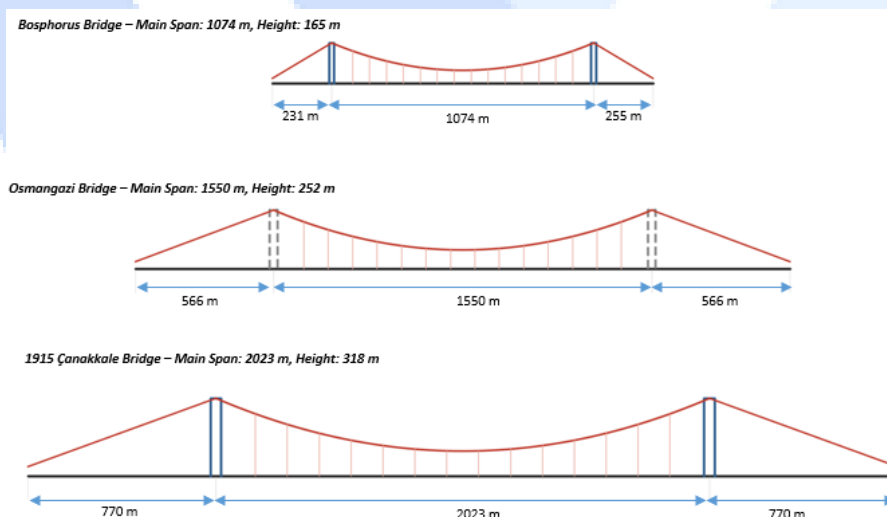


Figure 1. Elevation (side-view) comparison of the three suspension bridges, drawn to the same horizontal scale, illustrating the difference in span lengths

2.1. Finite Element Models of Suspension Bridges

To ensure a consistent basis for comparison, the three bridges were modeled in SAP2000 [11] using the same general finite element modeling strategy while preserving their individual geometric and structural characteristics. The towers, decks, main cables, and backstay cables were represented by frame elements, whereas the hangers were modeled as axial-force-carrying truss elements. The pre-strain values listed in Table 2 were introduced into the main and backstay cables to establish the required initial cable-force state. Geometric nonlinearity was considered during the equilibrium solution, and the structural configuration was established iteratively using the Newton–Raphson procedure. Regarding the boundary conditions, the tower bases and terminal cable anchorage points were idealized as fixed supports. Deck support and connection conditions were defined according to the structural configuration of each bridge, using roller-type restraints and link elements, as required, to represent deck-end, deck–tower, transition-pier, and cable–tower/saddle connections. Soil–structure interaction (SSI) was not explicitly modeled; no soil springs or deformable soil domains were included, and the foundation–fixed support conditions represented ground interfaces. The spring/link elements used at selected structural connections represent connection flexibility and do not represent soil flexibility.

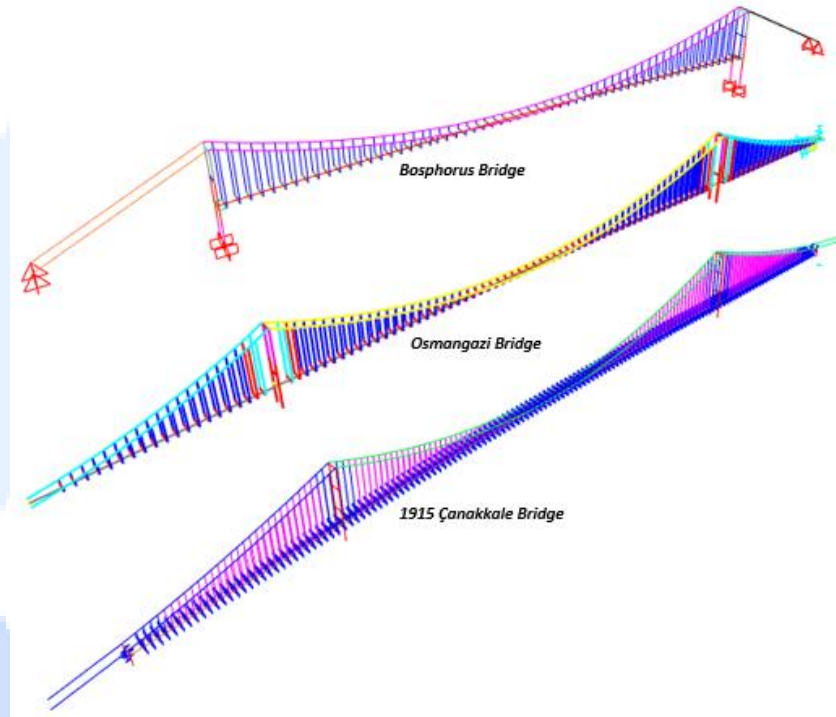


Figure 2. FE modeling of the three bridges

Table 2. Comparison of the FE properties of the three bridges

Parameter	Bosphorus Bridge	Osmangazi Bridge	1915 Çanakkale Bridge
Mesh size	17.5 m	25 m	25 m
Damping ratio (reference study)	2%	2%	2%
Number of modes considered	100	90	100
Main cable pre-strain	0.00276	0.00264	0.00260
Backstay cable pre-strain	0.0034	0.0033	0.0032

2.2. Comparison of Modal Analysis Results

The natural frequencies and corresponding mode shapes obtained from the finite element models were compared with experimental and/or numerical reference results reported in the literature for each bridge. Attention was given to matching modes according to their deformation characteristics rather than solely according to their numerical order, since closely spaced modes may occur in long-span suspension bridges. The differences between the calculated and reference natural frequencies were generally below 5%, indicating satisfactory agreement for the present comparative investigation. Figure 3 presents the first five identified mode shapes and their corresponding natural frequencies, while Figure 4 compares the frequencies of the first three comparable modal patterns among the three bridges.

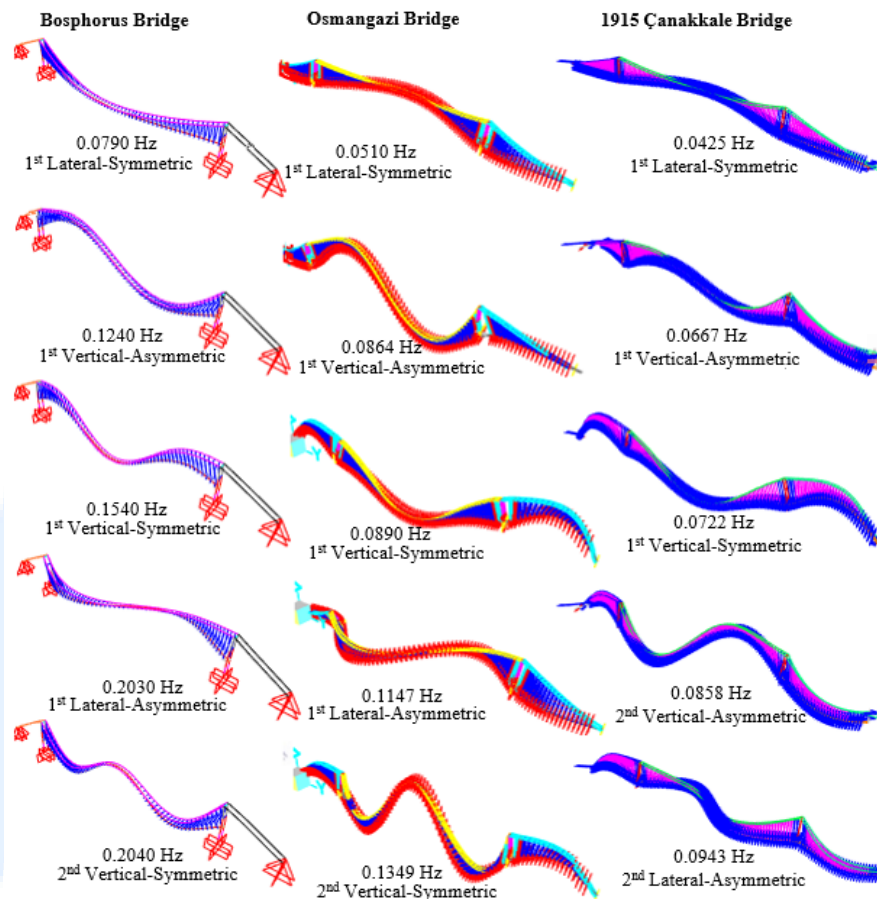


Figure 3. Frequency and mode shapes obtained from FE modeling of the three bridges

As shown in Fig. 4, as the main span length increases from 1074 m to 2023 m (an increase of approximately 88%), the first-mode frequency decreases from 0.079 Hz to 0.0425 Hz, a reduction of approximately 46%. This finding is consistent with the general trend expected for long-span suspension bridges, in which increasing span length results in a more flexible structure, a longer fundamental period, and a lower natural frequency. Furthermore, the fact that the first mode is lateral-symmetric in character for all three bridges indicates that the low-frequency behavior of this type of bridge is largely governed by the lateral flexibility of the deck-cable system.

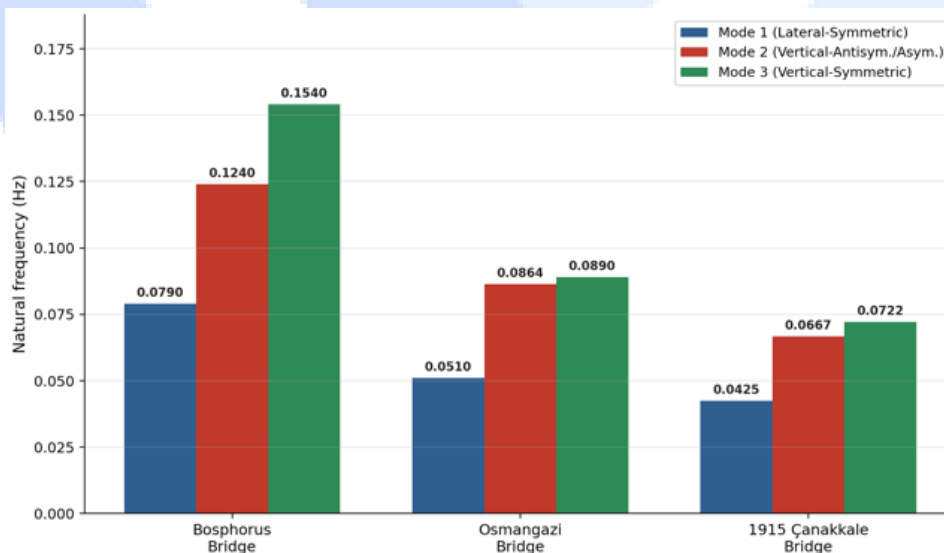


Figure 4. Comparison of the first three natural frequencies of the three bridges

2.3. Comparison of Seismic Analysis Results

The three-component acceleration record obtained at the Gebze station during the 17 August 1999 Kocaeli earthquake was used as a common seismic input for all three bridge models. The use of the same ground-motion record provides a controlled comparison in which differences in the calculated responses can primarily be associated with differences in the structural and modal characteristics of the bridges rather than with variations in earthquake input. Accordingly, the analyses are intended to investigate relative structural response under a common excitation and should not be interpreted as site-specific seismic safety assessments of the individual bridges. The EW, NS, and vertical components of the record are shown in Figure 5. Time-history analyses were performed for damping ratios of 2% and 5% to evaluate the sensitivity of the structural response to the assumed damping level. Tables 3, 4 and 5 show the values of the maximum internal forces obtained from the dynamic analysis performed considering a 2% and 5% damping ratio for Bosphorus Bridge, Osmangazi Bridge and 1915 Çanakkale Bridge, respectively.

Table 3. Maximum displacement and internal forces of the Bosphorus Bridge at 2% and 5% damping ratios

Parameter	Damping Ratio		Diff. (%)
	2%	5%	
Deck vertical displacement (cm)	23.96	17.34	27.63
Deck shear force (kN)	2264	2056	9.19
Deck bending moment (kNm)	279372	255913	8.40
Tower longitudinal displacement (cm)	24	12.01	49.96
Tower shear force (kN)	1237	1130	8.65
Tower bending moment (kNm)	71496	61026	14.64
Main cable axial force (kN)	122353	121849	0.41
Hanger axial force (kN)	1112	1101	0.99

Table 4. Maximum displacement and internal forces of the Osmangazi Bridge at 2% and 5% damping ratios

Parameter	Damping Ratio		Diff. (%)
	2%	5%	
Deck vertical displacement (cm)	68.41	62.04	9.31
Deck shear force (kN)	4873	4380	10.12
Deck bending moment (kNm)	874907	831430	4.97
Tower longitudinal displacement (cm)	22.76	17.33	23.86
Tower shear force (kN)	8025	7251	9.64
Tower bending moment (kNm)	513334	465203	9.38
Main cable axial force (kN)	204404	203660	0.36
Hanger axial force (kN)	2010	1952	2.89

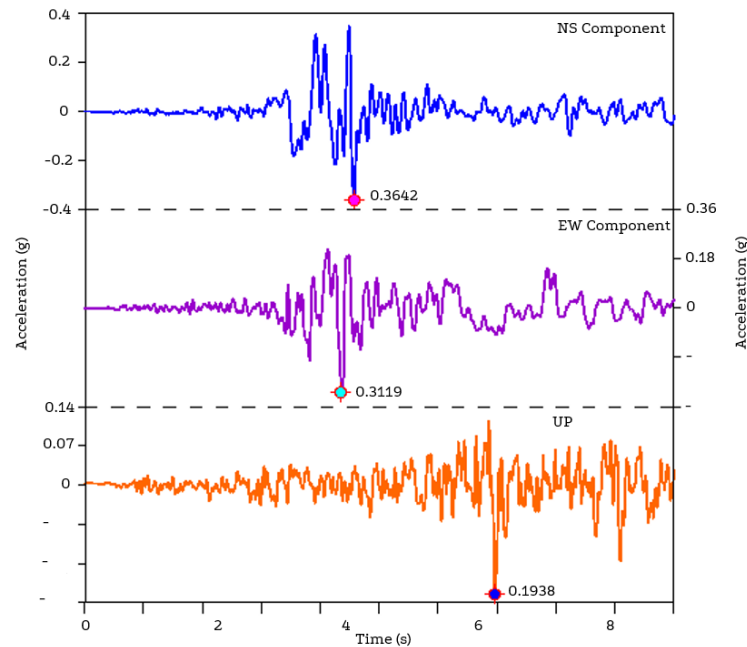


Figure 5. Acceleration components of the 1999 Kocaeli earthquake

Table 5. Maximum displacement and internal forces of the 1915 Çanakkale Bridge at 2% and 5% damping ratios

Parameter	Damping Ratio		Diff. (%)
	2%	5%	
Deck vertical displacement (cm)	14.89	12.71	14.64
Deck shear force (kN)	684	636	7.02
Deck bending moment (kNm)	34119	30344	11.06
Tower longitudinal displacement (cm)	20.37	17.99	11.68
Tower shear force (kN)	2902	2530	12.82
Tower bending moment (kNm)	261796	229669	12.27
Main cable axial force (kN)	286923	286910	0.005
Hanger axial force (kN)	2368	2359	0.38

Structural damping dissipates vibrational energy and reduces the amplitude of the dynamic response. Consequently, the response quantities obtained at $\zeta = 5\%$ are expected to be smaller than those obtained at $\zeta = 2\%$ for the same bridge subjected to the same ground motion. Figure 6 summarizes the percentage difference between the 2% and 5% damping results for each bridge.

Figure 6 compares the percentage reductions obtained when the damping ratio is increased from 2% to 5%. The influence of damping is considerably more pronounced for deck and tower response quantities than for the total axial forces in the main cables and hangers. Among all investigated parameters, the largest reduction is observed in the longitudinal displacement of the Bosphorus Bridge tower, which decreases by approximately 50%. For the remaining deck- and tower-related quantities, reductions are generally between approximately 5% and 15%, except for several displacement responses. In contrast, the variations in the total main-cable and hanger axial forces remain below approximately 3%. The physical implications of these differences and their relationship with span length and structural characteristics are discussed in Section 4.

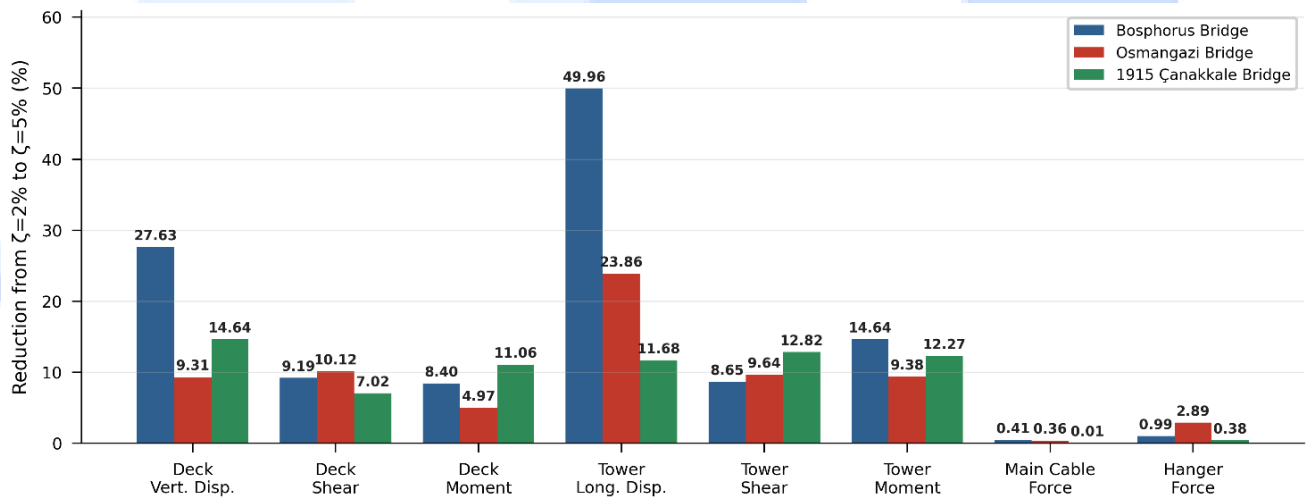


Figure 6. The percentage difference between the 2% and 5% damping results for each bridge

3. Discussion

The comparative modal results reveal a clear relationship between main-span length and the global dynamic characteristics of the three suspension bridges. As the main span increases from 1074 m for the Bosphorus Bridge to 1550 m for the Osmangazi Bridge and 2023 m for the 1915 Çanakkale Bridge, the fundamental frequency decreases from 0.0790 Hz to 0.0510 Hz and 0.0425 Hz, respectively. This corresponds to an approximately 46% reduction between the shortest- and longest-span bridges and an increase in the fundamental period from 12.66 s to 23.53 s. Despite the considerable differences in geometry and span length, the first three comparable modes preserve a similar sequence, consisting of lateral-symmetric, vertical-asymmetric, and vertical-symmetric deformation patterns. These results indicate that increasing span length primarily shifts the global vibration characteristics toward a lower-frequency range while preserving the general modal hierarchy of the suspension-bridge system.

In contrast to the systematic variation observed in the modal properties, the seismic response does not exhibit a monotonic relationship with span length. Under the common 1999 Kocaeli earthquake excitation, the Osmangazi Bridge develops the largest deck vertical displacement, deck shear force, deck bending moment, tower shear force, and tower bending moment among the three bridges. For example, at 2% damping, the maximum deck vertical displacement reaches 68.41 cm for the Osmangazi Bridge, compared with 23.96 cm for the Bosphorus Bridge and 14.89 cm for the 1915 Çanakkale Bridge. Similarly, the maximum tower bending moment of the Osmangazi Bridge reaches 513334 kNm, whereas the corresponding values are 71496 kNm and 261796 kNm for the Bosphorus and 1915 Çanakkale bridges, respectively. Therefore, span length alone cannot be used to predict the magnitude of seismic demand. The response is governed by the combined effects of structural stiffness, mass distribution, modal characteristics, and the frequency content of the imposed ground motion.

The response spectra shown in Figures 7 and 8 provide further insight into this behavior. The fundamental periods of the three bridges lie in the long-period region of the selected Gebze ground motion, where spectral acceleration is considerably lower than its short-period peak. The 1915 Çanakkale Bridge, with a fundamental period of approximately 23.53 s, lies furthest along this long-period spectral range. This is consistent

with the comparatively low deck displacement, shear force, and bending moment obtained for this bridge under the selected excitation. However, the response cannot be attributed to fundamental period alone, since higher-mode contributions and differences in the structural systems also influence the resulting internal forces. The particularly high response levels obtained for several components of the Osmangazi Bridge demonstrate the importance of the interaction between the modal characteristics of each bridge and the spectral characteristics of the earthquake excitation.

The damping comparison also demonstrates that the influence of structural damping is strongly response-dependent. Increasing the damping ratio from 2% to 5% reduces all deck- and tower-related response quantities considered in the study, although the magnitude of the reduction varies considerably among the bridges and response parameters. The most pronounced change occurs in the longitudinal tower displacement of the Bosphorus Bridge, which decreases by approximately 50%, while reductions in deck and tower force demands are generally in the range of approximately 5–15%. In contrast, the total axial forces in the main cables and hangers show only limited percentage variation, remaining below approximately 3%. This difference can be associated with the large gravity-induced pretension already present in the cable system, which dominates the total axial force and consequently reduces the relative contribution of damping-induced changes to the reported maximum force.

Overall, the results demonstrate that the influence of span length on suspension-bridge behavior is considerably clearer in the modal domain than in the seismic-response domain. Longer spans systematically produce lower natural frequencies and longer fundamental periods, whereas the corresponding earthquake-induced displacement and force demand depend strongly on the interaction between the structural characteristics and the excitation spectrum. Therefore, the present results should be interpreted as a controlled comparison of the three bridges under a common ground-motion input rather than as a site-specific seismic safety assessment. Extending the analysis to a broader set of earthquake records with different spectral characteristics would provide a more comprehensive assessment of the span-dependent seismic response of long-span suspension bridges.

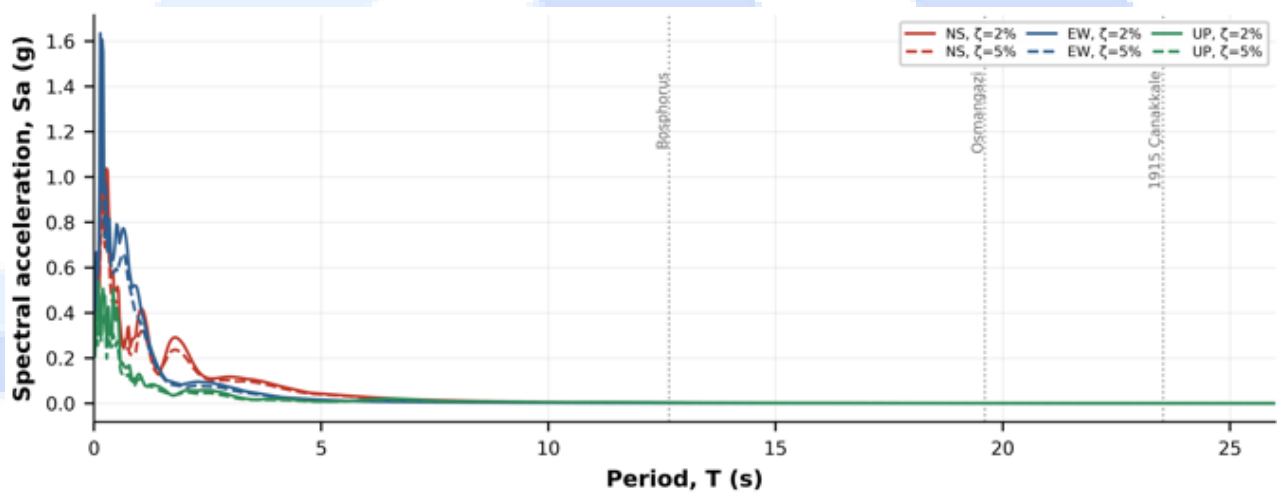


Figure 7. Elastic pseudo-acceleration (S_a) response spectra of the three components (NS, EW, UP) of the 1999 Kocaeli earthquake at the Gebze station, at 2% and 5% damping

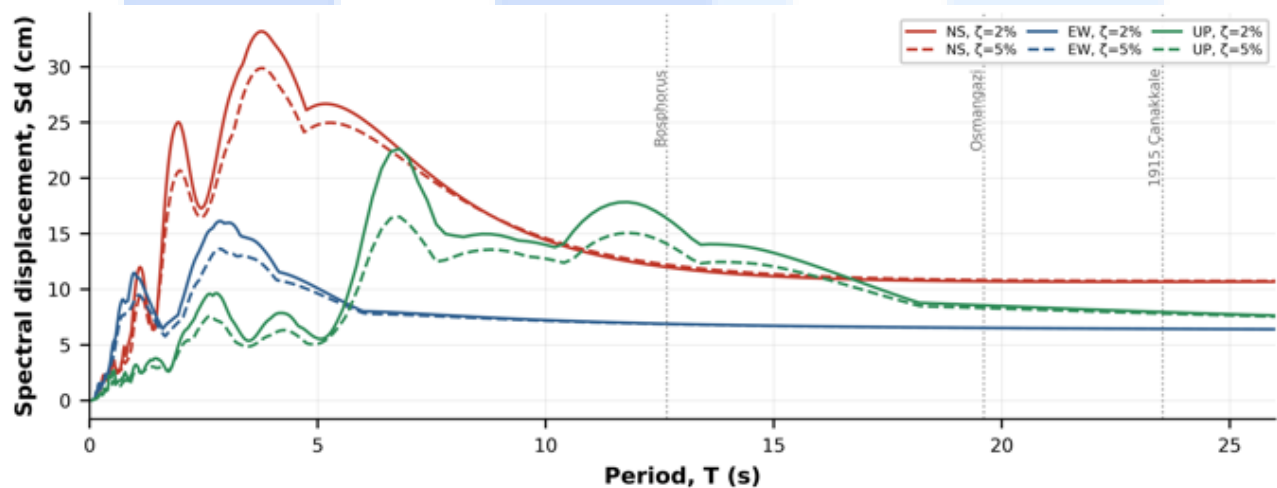


Figure 8. Elastic spectral displacement (S_d) response spectra of the three components (NS, EW, UP) of the 1999 Kocaeli earthquake at the Gebze station, at 2% and 5% damping

4. Conclusion

This study comparatively investigated the modal characteristics and seismic responses of the Bosphorus, Osmangazi, and 1915 Çanakkale suspension bridges, whose main spans range from 1074 m to 2023 m, using consistently developed three-dimensional finite element models. The comparison provides a controlled framework for examining how span length and the assumed damping ratio influence the dynamic behavior of long-span suspension bridges subjected to the same earthquake excitation.

A clear span-dependent trend was identified in the modal characteristics. As the main span increased by approximately 88% from the Bosphorus Bridge to the 1915 Çanakkale Bridge, the fundamental frequency decreased from 0.0790 Hz to 0.0425 Hz, corresponding to an approximately 46% reduction and an increase in the fundamental period from 12.66 s to 23.53 s. Despite these substantial frequency shifts, the comparable low-order modes retained a similar sequence of lateral-symmetric, vertical-asymmetric, and vertical-symmetric deformation patterns.

The seismic results, however, showed that earthquake demand does not vary monotonically with span length. The Osmangazi Bridge produced the largest response for several key parameters, including deck displacement, deck shear and bending moment, and tower shear and bending moment, whereas the 1915 Çanakkale Bridge exhibited comparatively lower deck-level demands under the selected Gebze ground motion. This demonstrates that seismic response is controlled not by span length alone but by the interaction between the structural modal properties and the frequency content of the excitation.

Increasing the damping ratio from 2% to 5% consistently reduced the deck- and tower-related dynamic responses, with reductions ranging from approximately 5% to 50% depending on the bridge and response quantity. The main cable and hanger axial forces were considerably less sensitive to the damping ratio in terms of their total maximum values, with differences generally remaining below 3%, owing to the dominant contribution of gravity-induced pretension.

The findings therefore indicate that increasing span length has a systematic and predictable influence on the modal characteristics of suspension bridges, whereas its influence on seismic demand is more complex and excitation-dependent. The use of a common earthquake record enables the structural differences among the bridges to be isolated; however, the results should not be interpreted as a direct ranking of their seismic safety. Future studies incorporating multiple ground motions with different spectral characteristics would further clarify the relationship between span length, damping, and the seismic response of ultra-long-span suspension bridges.

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Static Bending Analysis of FG-Plates with Porosity Using High Order Shear Theory

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Keywords

FG Plate,

Bending,

Porosity,

High order theory.

Abstract

The paper examines a bending of porous functional graded plate based on high- order shear theory. The proposed high-order shear deformation theory, as opposed to other theories, includes four unknowns. The technique of Navier is used in closed-form FG plate solutions. Results of deflections and stresses are presented. The Effects of different parameters, gradient index, thickness and porosity of FG plates, are discussed.

1. Introduction

These materials are generally recognized and produced in many foreign science labs, and one of the most widely used composites gives various fields such as engineering, the aeronautics, chemical, nuclear energy, electronics, optics, civil engineering, biomaterials, etc. [1-4]. The materials used in this field are functionally graded materials (FGMs). The material compositions of FGMs are assumed to vary smoothly and continuously in all gradient directions. In the mid-1980s, Japanese scientists developed the earliest FGMs as high-temperature-resistant materials for aerospace applications. During the sintering process and in the production of FGM porosity, materials may occur. This porosity is because of the large temperature coagulation difference between the material components [5]. The porosities in ref [6] are identified in lateral FGM samples with multi-stage sequence filtration. It is thus critical that the porosity effect is taken into account when designing FG dynamic load components.

The literature reviews on porosity effect are presented by several researchers in the recent works on porosity effect of FGM plates and beams. The effect of porosity is therefore important to consider in the design of static and bending plates made of functionally graded materials FGM [7-10] and sandwich plates FGM with porosity Usage [11] of a high-order shear-deformation theory. Consequently, Studies focused more and more on the dynamic and free vibration of plate made of functionally graded FGM materials in recent years [12-19].

This paper aims to present a research solution for the performance of static porosity-grading plates analysis using high-order theory. During the manufacture of these plates, defects such as porosities may appear, considering the porosities that can occur inside the functionally graded materials (FGM) plates. The Navier solution in closed-form is used to correct the limit shear correction factors on the plates and follows the equilibrium conditions of the porous FG layer. The results of the current theory of the porous FGM plates are studied as well as the effect of the thickness ratio, the scaled exponent factor, and the deflection and stress porosity. In the assessment of deflations and stress distribution of functionally classified material plates, the impact of porosity on shave deformation has been increased.

2. Theory and Formulations

Consider a rectangular, wide FG-platform with length (a), width (b) and thickness (h) of a given functional material as shown in figure 1.

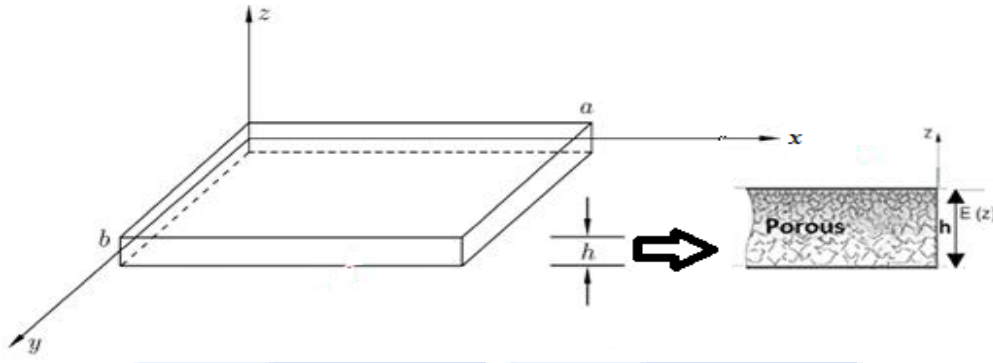


Figure 1. Geometry of FG porous plate

The FGM plate is subjected to a transverse load $q(x, y)$ for deformation analysis of the plate and a rectangular cartesian X and Y co-ordinates are attached. First and foremost, we will look at a non-homogeneity material with a porosity volumetric function, namely, ξ ($0 \leq \xi \leq 1$).

The functional relation for the ceramic and metal FGM plates between $E(z)$ is assumed.

$$E(\xi) = (E_c - E_m) \left(\frac{1}{2} - \frac{\xi}{b} \right)^p + E_m - (E_c + E_m) \frac{\xi}{2} \quad (1)$$

When E_c and E_m are the corresponding ceramic and metal elements and "p" is an exponent volume fraction that takes values greater or equal to zero. The above power law theory demonstrates a simple blending rule used to achieve the efficient characteristics of a ceramic metal platform.

The theory of shear deformation plates is ideal for displacements in this study:

$$\begin{aligned} u(x, y, \xi) &= u_0(x, y) - \xi \frac{\partial w_b}{\partial x} - f(\xi) \frac{\partial w_s}{\partial x} \\ v(x, y, \xi) &= v_0(x, y) - \xi \frac{\partial w_b}{\partial y} - f(\xi) \frac{\partial w_s}{\partial y} \\ w(x, y, \xi) &= w_b(x, y) + w_s(x, y) \end{aligned} \quad (2)$$

In the present theory, this function $f(\xi)$ is considered:

$$f(\xi) = \xi - \sin\left(\frac{\pi \xi}{b}\right) + \frac{\pi}{2b} \xi \quad (3)$$

3. Analytical Solutions FG-Plate

On the side edges of the FG plate are placed the following easily endorsed boundary conditions:

$$v_0 = w_b = w_s = \frac{\partial w_b}{\partial y} = \frac{\partial w_s}{\partial y} = N_x = M_x^b = M_x^s = 0 \quad \text{and} \quad x = 0, a \quad (4a)$$

$$u_0 = w_b = w_s = \frac{\partial w_b}{\partial x} = \frac{\partial w_s}{\partial x} = N_y = M_y^b = M_y^s = 0 \quad \text{and} \quad y = 0, b \quad (4b)$$

The external force can be expressed as per the Navier solution:

$$q(x, y) = \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} q_{mn} \sin(\lambda x) \sin(\mu y) \quad (5)$$

where $\lambda = m\pi/a$ and $\mu = n\pi/b$, « m » and « n » are mode numbers. Where q_0 represents the intensity of the load at the plate center.

For the case of a sinusoidal distributed load, we have

$$Q_{11} = q_0, m = n = 1 \quad (6)$$

We consider the following solution form (u, v, w_b, w_s), following the Navier solution protocol, to satisfy the boundary conditions

$$\begin{Bmatrix} u \\ v \\ w_b \\ w_s \end{Bmatrix} = \begin{Bmatrix} U_{mn} \cos(\lambda x) \sin(\mu y) \\ V_{mn} \sin(\lambda x) \cos(\mu y) \\ W_{bmn} \sin(\lambda x) \sin(\mu y) \\ W_{smn} \sin(\lambda x) \sin(\mu y) \end{Bmatrix} \quad (7)$$

Where the parameters U_{mn} , V_{mn} , W_{bmn} and W_{smn} are arbitrary. First order equations system as:

$$[K]\{\Delta\} = \{F\} \quad (8)$$

where the $\{\Delta\}$ and $\{F\}$ columns are referred to

$$\{\Delta\}^T = \{U_{mn}, V_{mn}, W_{bmn}, W_{smn}\}, \{F\}^T = \{0, 0, -q_{mn}, -q_{mn}\} \quad (9)$$

$$[K] = \begin{bmatrix} a_{11} & a_{12} & a_{13} & a_{14} \\ a_{12} & a_{22} & a_{23} & a_{24} \\ a_{13} & a_{23} & a_{33} & a_{34} \\ a_{14} & a_{24} & a_{34} & a_{44} \end{bmatrix} \quad (10)$$

The elements $a_{ij} = a_{ji}$ of the coefficient matrix $[K]$.

4. Results and Discussions

Numerical results for changes and stresses in functionally graded porosity plates are given in this section in order to confirm the exactness of the present formulation. The material properties of the FG plates are aluminum and alumina:

- Metal (Aluminum, Al): $E_m = 70$ GPa, $\nu = 0.3$.
- Ceramic (Alumina, Al_2O_3): $E_c = 380$ GPa, $\nu = 0.3$.

The following non-dimensional parameters are used for simplicity:

$$\bar{w} = \frac{10bE_c}{a^2q_0} w \left(\frac{a}{2}, \frac{b}{2} \right), \bar{\sigma}_x = \frac{10b^2}{a^2q_0} \sigma_x \left(\frac{a}{2}, \frac{b}{2}, \frac{b}{2} \right), \bar{\tau}_{xy} = \frac{b}{aq_0} \tau_{xy} \left(0, \frac{b}{2}, 0 \right) \quad (11)$$

Thicknesses coordinate $\bar{z} = z / h$.

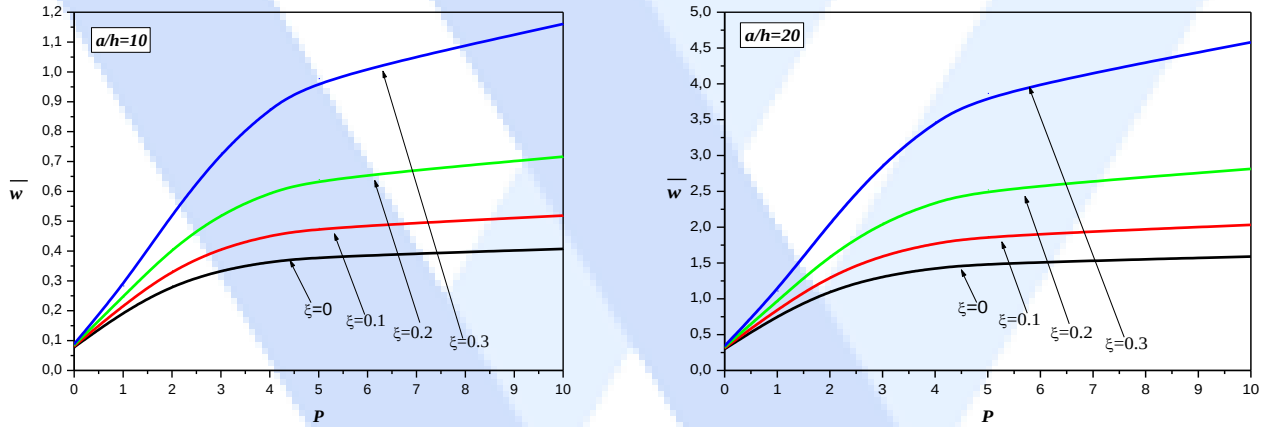


Figure 2. Displacement variation as a function of the power index “p” and the volume fraction of porosity of a FG plate

Figure 2 shows non-dimensional displacement differences for two thickness ratios ($a/h=10$ and $a/h=20$) based on the power and law index “p” for the perfect floor ($\xi = 0$) and the three porosity values ($\xi = 0.1, 0.2$ and 0.3). From this curve, it can be concluded that the dimensional transition is the with the rise in the force law index. Moreover, increasing the porosity coefficient increases the dimensional displacement of the two thickness ratios.

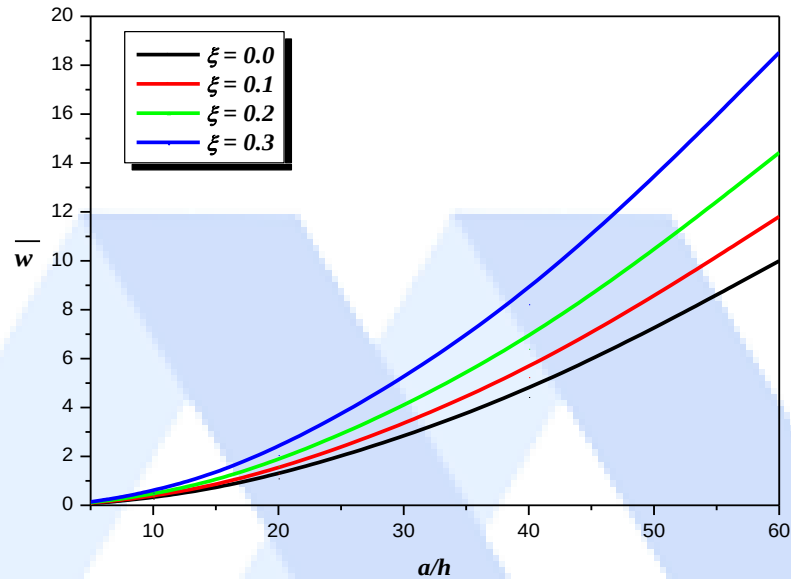


Figure 3. Displacement Variation depending on the thickness ratio for various porosity factor values ξ

Figure 3 illustrates the influence of the aspect ratio (a/h) for the fluidization of the FG plate for perfect plate size ($\xi = 0$), and three values of the imperfect plates of the porosity coefficient ($\xi = 0.1, 0.2$ and 0.3). It is assumed the power-law index to be constant, $p = 2$. The increase in dimensional displacements, explained through the effect of material rigidity, i.e., the rise of the value of the porosity coefficient (ξ) is seen in this figure, leads to an increase in plate displacements. The impact of porosity on the bending of a plate has also been shown to increase with the larger values of the ratio of thickness to length.

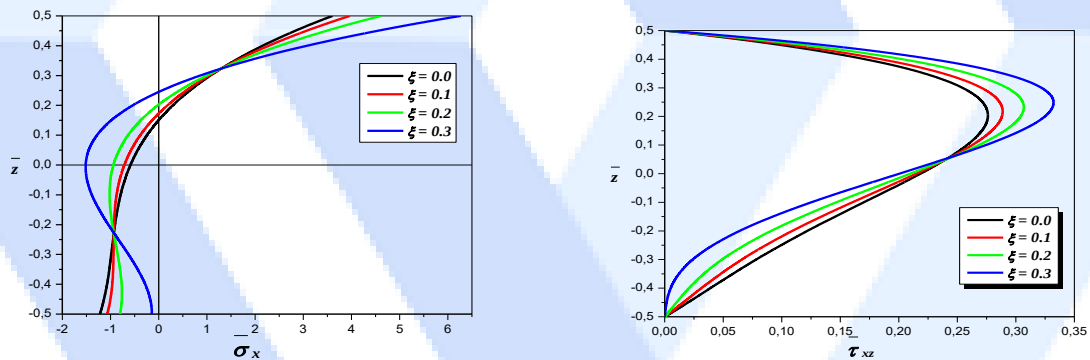


Figure 4. The thickness of FG P axial stress (a) and shear stress (b) distribution for different porosity factor ($a/h=10$ and $p=2$)

Figure 4a took account of the effect of the porosity of the FGP by adding the coefficient (ξ). Therefore, four values are maintained ($\xi = 0, 0.1, 0.2$ and 0.3). The increase in the porosity index (ξ) can be observed to contribute to increased stress. This can be explained by the reduction of the rigidity of the plate by the porosity. The stresses are tensile above the middle plane and compressed under the middle plane. It should be remembered that the overall stress based on the magnitude of the volume fraction exponent. Figure 4b indicates shear stresses through the transverse thickness distribution. This figure shows the remarkable direct impact of the porosity effect; this occurs at a point in the FGP middle plane and lowers transverse shear stress.

5. Conclusion

The paper is intended to analyses the bending effect of functionally plates with porosities that can occur inside of this type of plates based on the concept of high order theory. This paper supports study and the analysis of various parameters, such as the material parameter, surface thickness and coefficient of porosity. The control equations are resolved with the Navier-style closed-form solution for sinusoidal FG plates. From this work, we can say that the current and simple theory of the resolution of the mechanical behavior of porosity FGM plates that defect factories.

Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Strategies for the Use of Lignosulphonates in Asphalt Modification and Their Effects on Performance

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Keywords

Asphalt modification,

Lignosulphonate,

Rheological behaviour,

Composite systems,

Sustainable pavement.

Abstract

Lignosulphonate-based biopolymers present significant performance trade-offs between high-temperature stability and low-temperature flexibility in asphalt modification. In this study, modification strategies were examined under three main categories: direct use, purification, and hybrid/composite systems. Analyses show that the type of cation in the modifier (calcium, sodium, magnesium) directly determines the nature of the bond formed with the bitumen and the resulting rheological behaviour. While the direct use of existing derivatives increases rutting resistance, it leads to low-temperature brittleness and phase separation; purification and composite approaches, on the other hand, require complex and costly processes on an industrial scale. To overcome these bottlenecks, the use of potassium and ammonium lignosulphonates—which have not yet been investigated in the literature—has been identified as a critical research gap.

1. Introduction

Asphalt, the primary binding material in flexible pavements, is subject to various forms of deterioration—such as rutting, fatigue cracks and thermal-oxidative ageing—due to the effects of environmental factors including heavy vehicle traffic, increasing axle loads and climate change. Modifying the rheological and mechanical properties of the binder has become an industrial necessity in order to prevent these performance losses, extend the service life of the pavement and reduce maintenance costs. Today, synthetic polymers such as styrene-butadiene-styrene (SBS) and ethylene vinyl acetate (EVA), nanomaterials, rubber derivatives and various chemical anti-stripping agents are widely used in asphalt modification [1], [2], [3], [4]. The high costs and environmental footprints of traditional petroleum-based and synthetic polymer modifiers have led researchers to focus on renewable and sustainable materials. In line with the objectives of sustainable pavement engineering, the use of biopolymers derived from industrial or agricultural waste in asphalt modification has gained significant momentum. These renewable, non-toxic and low-cost biopolymers not only enhance the ageing resistance of asphalt but also reduce dependence on fossil resources, thereby making a critical contribution to the circular economy and environmental sustainability [5], [6], [7], [8].

Lignin, the second most abundant biopolymer in nature after cellulose, is the largest industrial by-product of the paper and pulp industry [9]. Lignosulphonates, obtained by rendering lignin water-soluble via the sulphite pulping process, transform into a complex polymer network that simultaneously contains both hydrophilic and hydrophobic (amphiphilic) functional groups in their structure, compared to standard types of lignin [10], [11], [12]. This amphiphilic character enables lignosulphonates to exhibit significantly higher solubility and thermodynamic compatibility within the non-polar asphalt matrix [12], [13], [14]. The phenolic hydroxyl groups and benzene rings, which are densely present in their molecular structures, act as a powerful antioxidant shield by scavenging free radicals, thereby slowing down the thermal-oxidative degradation of asphalt [15], [16], [17]. Simultaneously, these aromatic structures mitigate the photo-oxidative ageing of asphalt through their ability to chemically absorb UV rays [17], [18], [19], [20]. Combining high matrix compatibility with versatile anti-ageing protection mechanisms, lignosulphonates stand out as a high-performance and environmentally friendly modifier that extends the service life of flexible pavements [9], [15], [21], [22].

However, the effectiveness of lignosulphonates in asphalt modification; depends on the type of cation they contain (calcium, sodium, magnesium) and the preferred integration strategy (direct incorporation, purification or composite hybridisation), leading to significant thermomechanical incompatibilities between high-temperature stability and low-temperature flexibility. In this context, the present review study aims to synthesise the thermodynamic, chemical and rheological behaviour dynamics of lignosulphonate-based biopolymers within the asphalt matrix from a holistic perspective. Structural bottlenecks—such as phase separation, low-temperature brittleness and rheological incompatibility—caused by calcium, sodium and magnesium lignosulphonate applications, which have been predominantly investigated in the literature, have been systematically examined. In light of these analyses, the conceptual and academic framework for the research gap concerning potassium and ammonium lignosulphonate derivatives has been established.

2. Classification by Lignosulphonate Type and Incorporation Strategy

The existing literature clearly demonstrates that lignosulphonates cannot be assessed using a single standard approach in asphalt modification. The chemical structure of the modifier (cation type) and the method of incorporation into the mixture have a direct and significant effect on the final thermomechanical performance of the modified bitumen. In particular, when specific parameters such as purity and melting consistency are taken into account in experimental bitumen analyses, whether the modifier is alkali- or alkaline earth metal-based shapes the material's entire rheological behaviour. In line with these structural and functional dynamics, the approaches in the literature have been classified under three fundamental modification strategies: direct use, purification processes, and hybrid/composite systems.

2.1. Direct Utilization Strategies and The Cation Effect

The modification strategy, in which lignosulphonate powder is added directly to hot bitumen using high-shear mixers without undergoing any chemical pretreatment or purification process, stands out for its simplicity of production and cost advantages. In the literature, the different bonding mechanisms established between calcium, sodium and magnesium lignosulphonates and bitumen, and the associated rheological behaviour, have been investigated primarily under this strategy [9], [23], [24], [25], [26].

Calcium lignosulphonate (CLS) increases the cohesion of the soft asphalt matrix (SAM) by forming strong covalent bonds with the polar components of bitumen (particularly resins and asphaltenes) via the hydroxyl (-OH) groups in its molecular structure [23]. These chemical reactions significantly harden the material by increasing the asphaltene content within the bitumen, even before ageing has taken place. This increase in the asphaltene content maximizes the mixture's resistance to rutting at high temperatures, whilst also enabling CLS, as an oxidation inhibitor, to slow down the thermal-oxidative ageing process [23], [24]. However, this hardening mechanism, which is based on covalent bonds, causes the asphalt to become brittle at low temperatures and reduces its fatigue life. Research indicates that the threshold usage rate for CLS, which optimally balances resistance to rutting and fatigue, is 15 per cent; higher dosages (for example, 20 per cent) increase brittleness and reduce crack resistance [11], [13], [24], [27].

On the other hand, when **sodium lignosulphonate (SLS)** is added directly to bitumen, it adheres to the matrix via hydrogen bonds and weak physical interactions rather than forming new covalent bonds [26]. In the case of magnesium lignosulphonate (MLS) added to bitumen, although no chemical reaction or formation of covalent bonds is observed, a strong electrostatic interaction mechanism comes into play. Fourier-transform infrared spectroscopy (FT-IR) analyses have demonstrated that no new absorption peak is formed in the spectrum of pure asphalt. However, the significant decrease observed in the peak areas, particularly in the 1600 cm^{-1} band, and the disappearance of the -OH stretches associated with MLS following a short-term ageing (RTFOT) test clearly demonstrate that this strong electrostatic attraction has been established between the bitumen's hydrocarbon chains and MLS [25].

This electrostatic attraction force generated by MLS significantly improves the thermal stability and resistance to tyre rutting of the modified asphalt by shifting the degradation curves to higher temperatures [20], [25]. From a rheological perspective, the addition of MLS facilitates the material's viscous flow by reducing the activation energy (E_a) required to overcome the forces inhibiting intermolecular slip, thereby helping it to retain its flow characteristics despite ageing [20]. However, Scanning Electron Microscopy (SEM) analyses have revealed that increasing MLS dosage (particularly at 8 per cent and above) disrupts the dispersion equilibrium within the bitumen and leads to agglomeration in the form of angular aggregates, causing undesirable deviations in penetration and softening point values [25].

2.2. Purification Processes (Sodium-Based Approaches)

Although sodium lignosulphonate (SLS) is an environmentally friendly biopolymer with the potential to enhance the ageing resistance of asphalt, it must be incorporated into asphalt at high levels due to its low modification efficiency. Unlike calcium lignosulphonates, SLS forms weak physical interactions in pure bitumen, and the complex, heterogeneous composition inherent to lignin leads to instability in the final properties of the modified bitumen. Whilst increasing the SLS content has a positive effect on the ageing resistance of the modified bitumen, these high dosages directly impair the material's low-temperature cracking resistance and flexibility. Therefore, integrating SLS into the bitumen following molecular-level purification (homogenization) emerges as an essential strategy to preserve low-temperature performance whilst reducing the need for high dosages, and to maximize the material's antioxidant efficacy.

In the literature, the step-wise solvent fractionation method—in which water is used as the solvent and ethanol as the anti-solvent—is the preferred method for purifying sodium lignosulphonate. This physicochemical separation process reduces the polymer distribution index (PDI), separating SLS into five distinct homogeneous fractions (F1–F5) with a much narrower and more concentrated molecular weight distribution compared to the raw material [28].

Upon examining the chemical reactivity of the fractions obtained as a result of the purification process, it was found that the fractions with low molecular weight (particularly F4 and F5) contained a much higher concentration of phenolic hydroxyl groups capable of trapping free radicals within their molecular structures. This endows these fractions with exceptional DPPH radical scavenging capacity and antioxidant activity. However, due to its very low molecular weight and density, the F5 fraction rises to the surface in hot bitumen, causing serious storage stability and phase separation (segregation) problems [28].

In contrast, the F4 fraction, which constitutes approximately 32 per cent of the original lignin, exhibits excellent surface compatibility with bitumen and is able to disperse homogeneously within the mixture. The F4 fraction strongly inhibits the oxidation reactions (particularly the chain growth stage) during the bitumen ageing process. Results from Dynamic Shear Rheometry (DSR) and Thin Film Oven Test (TFOT/PAV) demonstrate that the purified F4 fraction provides maximum resistance to both short- and long-term thermal-oxidative ageing without reducing the original Performance Grade (PG grade). TLC-FID analyses also demonstrate that aged F4-modified bitumen reduces the conversion rate to heavy components (asphaltenes and resins) by half [28].

2.3. Composite and Hybrid Systems: Integration with Polymers and Nanomaterials

In order to overcome the brittleness and compatibility issues caused by the use of lignosulphonates on their own (in their raw form) at low temperatures, hybrid systems reinforced with polymers, nanomaterials and bio-oils have been developed, as reported in the literature.

2.3.1. Polyester Fiber-Reinforced Calcium Lignosulphonate Composites

It is known that calcium lignosulphonate (CLS) increases cohesion in the bituminous matrix (SAM) by forming covalent bonds [29], [30], [31]. However, the addition of polyester fibers (PEF) to the system has been investigated with the aim of reducing the low-temperature brittleness, fatigue sensitivity and TDC (Top-to-Bottom Cracking) caused by CLS [30], [31], [32]. In their research, in which Ziaee et al. utilized the SCB (Semi-Circular Bending) test, it was demonstrated that the combined use of 0.25 per cent polyester fibers with 5–10 per cent calcium lignosulphonate significantly increased the Fracture Energy (G_f) and Fracture Toughness (K_{IC} , K_{IIC}) values under both Short-Term and Long-Term Ageing (LTA) conditions, as well as in Type I (tensile) and Type II (shear) fracture modes [29], [30], [32]. Furthermore, in optimization studies conducted using the balanced mixture design (BMD) principle and utilizing IFIT, IDEAL-CT and Dynamic Creep tests, it was determined that the optimum CLS content was 15 per cent under both aged and unaged conditions [33]. Whilst polyester fibers enhance interlocking by forming bridges between aggregates, CLS inhibits chemical oxidation; this synergy optimizes the modified asphalt's resistance to elastic deformation by increasing the Tensile Stiffness Index (TSI) and Tensile Strength (TS) [29], [30], [31].

2.3.2. Double-Layered Hydroxide (LDH)-Lignosulphonate Hybrid Composites

To resolve the agglomeration problem of inorganic double-layered hydroxides (LDHs) in asphalt, lignosulphonates have been used as a technological interfacial agent in three different forms [15], [17], [18], [22]:

- **Intercalation:** By intercalating the lignosulphonate between LDH layers (LS-LDHs), the d-spacing in XRD analyses was widened from 0.76 nm to 2.15 nm, ensuring the material's homogeneous dispersion within the bitumen and enhancing its storage stability (ΔS). This structure combines the physical UV reflectivity of LDH with the chemical UV absorbency of lignin [18], [22].
- **Grafting:** The hydroxyl groups of lignosulphonates were covalently grafted onto LDH via diisocyanates (MDI, TDI, IPDI) (LS-g-LDHs). This cross-linked, stable network has further improved photo-oxidative ageing resistance compared to pure LDH by suppressing the increase in carbonyl and sulphoxide groups resulting from ageing, as observed in FTIR data [17].
- **Coating:** The phenolic hydroxyl-rich fractions (F4 and F5) of purified sodium lignosulphonate were used to coat LDH directly (@LDHs). This strategy significantly increased hydrophobicity (by up to 43 per cent) and created an excellent synergistic shield that delays thermal and UV ageing by trapping free radicals [15].

2.3.3. Chemically Modified Lignosulphonate and Bio-Oil Composites

Chemical acetylation modification is applied to overcome the compatibility issues caused by the hydrophilic nature of lignin-based materials. Acetylation has increased the water contact angle from 0° to 48.6° by converting the hydroxyl groups in sodium lignosulphonate into aliphatic ester bonds. To counterbalance the additional stiffness and low-temperature brittleness caused by this process, pine needle oil – a bio-oil rich in terpenoids and aromatics – was used. BBR and DSR test results show that pine needle oil improves both the high-temperature rutting resistance and the low-temperature cracking performance of the modified asphalt simultaneously by reducing the interfacial tension [34].

2.3.4. Waste Plastic (LDPE)-Lignin Polymer Composites

Lignin was used as a compatibilising agent to address the poor compatibility between polar bitumen and non-polar waste low-density polyethylene (LDPE). SEM analyses showed that plastic agglomeration was eliminated in the presence of lignin. The addition of lignin was demonstrated to form Van der Waals bonds between the bitumen and LDPE, as evidenced by shifts in the FTIR spectrum from 1041.77 cm^{-1} and the Marshall Stability of the mixture with an optimum lignin addition of 1 per cent increased by 89.5 per cent (from 1,781.40 kg to 3,375.92 kg) compared to pure asphalt [35].

3. Comparison of Mixture Performance by Modification Type

The modification of bituminous binders with lignosulphonate-based biopolymers exhibits different rheological and microstructural behaviour depending on the chemical integration strategy employed. In the light of current literature data, the phase interactions within the asphalt matrix, the underlying mechanisms of action and their implications for the final pavement performance resulting from these modification techniques are systematically presented in Table 1.

Table 1. Mechanisms of action and performance evaluation of lignosulphonate modification strategies

Modification Type	Primary Impact Mechanism	Advantages	Disadvantages	References
Direct (Raw) Utilization (CLS, SLS, MLS)	Enhancement of matrix cohesion via covalent bond formation (CLS) or physical/electrostatic interactions (SLS, MLS).	Increased high-temperature rutting resistance and complex shear modulus (G^*); high moisture damage resistance (TSR > 80%).	Low-temperature embrittlement, reduction in fatigue life; agglomeration and phase separation at high dosages.	[9], [12], [13], [14], [20], [23], [24], [25], [26], [27], [36], [37]
Coupling Agent Integration	Establishment of chemical bridging (Van der Waals forces) between the polar structure of asphalt and the non-polar nature of waste plastics (LDPE).	Elimination of thermodynamic phase incompatibility; significant increase (89.5%) in Marshall stability.	Complex mix design of the polymer matrix; difficulty in the homogenization of waste plastics within the asphalt binder.	[35]
Fiber Hybrid Systems (Lignosulfonate + PEF)	Microscopic bridging between aggregates provided by fibers coupled with the chemical reinforcement of the matrix by lignosulfonate.	Enhancement in Fracture Energy (G_f) and fracture toughness under Mode I and Mode III loading; high resistance to freeze-thaw damage (FTD).	Workability challenges at high fiber and additive dosages; high sensitivity in mix optimization.	[29], [31], [32], [38]
Inorganic-Organic Nanohybrids (LS-g-LDHs)	Covalent grafting or interlayer intercalation of lignosulfonate into layered double hydroxides (LDH).	Outstanding UV aging resistance (RPR, SPI, VAI) driven by the synergy of inorganic physical reflection and chemical absorption.	Nanomaterial dispersion and agglomeration issues; high synthesis and reaction costs.	[15], [17], [18], [21], [22]
Purification and Bio-Oil Hybrids	Improvement of hydrophobicity via acetylation and restoration of the light molecular balance utilizing bio-oils (e.g., pine needle oil).	Improved low-temperature flexibility (m-value); superior mitigation of TFOT and PAV oxidative aging (particularly in the F4 fraction).	Complex and costly purification processes (e.g., solvent extraction via ethanol); barriers to industrial-scale implementation.	[16], [34]

4. Conclusions and Evaluation

The use of lignosulphonate-based biopolymers to enhance the performance of asphalt pavements against environmental factors and traffic loads holds great potential for sustainable pavement engineering. In light of the comprehensive literature review carried out, the following key conclusions regarding asphalt modification practice have been reached:

- The direct (raw) use of calcium, sodium and magnesium lignosulphonates significantly improves the binder's high-temperature rutting resistance and ageing tolerance. However, this approach creates thermomechanical drawbacks such as embrittlement at low temperatures, rheological incompatibility and agglomeration at higher dosages.
- Solvent-based purification (fractionation) processes, inorganic nanomaterial doping (LDH) and fiber/bio-oil-reinforced composite systems, proposed in the literature to address these shortcomings, have yielded highly successful results at laboratory scale. However, these methods require production processes that are complex, costly and difficult to optimize at an industrial scale.

The true technological breakthrough in asphalt modification lies in the development of binders capable of ensuring matrix compatibility and rheological balance through direct application, without the need for numerous chemical treatments. A comprehensive literature review has revealed that potassium and ammonium lignosulphonates have not yet been investigated in the context of asphalt modification. It has been determined that the extent of the interaction between the potassium and ammonium salts present in the lignosulphonate structure and the asphalt matrix will be an interesting research topic for future studies aimed at filling this obvious gap in the literature. Furthermore, in order to fully validate the industrial applicability of potassium and ammonium lignosulphonates, future research must go beyond macroscopic rheological evaluations. It is of vital importance to elucidate the microstructural interaction mechanisms between these new salts and the asphalt matrix using advanced spectroscopic and morphological techniques (e.g. FTIR, SEM, XRD). Furthermore, to transfer these sustainable binders from the laboratory to full-scale road paving engineering, it will be vital to assess their long-term antioxidant potential under a combination of hydrothermal and UV ageing conditions, followed by comprehensive Life Cycle Assessments (LCA) and field trials.

Nomenclature

SBS: Styrene-butadiene-styrene
EVA: Ethylene vinyl acetate
CLS: Calcium lignosulphonate
SLS: Sodium lignosulphonate
MLS: Magnesium lignosulphonate
FT-IR: Fourier transform infrared spectroscopy
RTFOT: Rotational thin film oven test
SEM: Scanning electron microscopy
 E_a : Activation energy
PDI: Polymer distribution index
DSR: Dynamic shear rheometry
TFOT: Thin film oven test
PAV: Pressure ageing vessel
PG: Performance grade
TLC-FID: Thin layer chromatography - flame ionization detector
SAM: Bituminous matrix
PEF: Polyester fibers
TDC: Top-to-bottom cracking
SCB: Semi-circular bending
BBR: Bending beam rheometer (test type)
 G_f : Fracture energy
 K_{Ic}, K_{IIc} : Fracture toughness
STA-LTA: Short-term and long-term ageing
BMD: Balanced mixture design
TSI: Tensile stiffness index
TS: Tensile strength
I-FIT: Illinois flexibility index test
IDEAL-CT: Indirect tensile asphalt cracking test
LDHs: Double-layered hydroxides
LS-LDHs: Lignosulphonate between LDH layers
 ΔS : Storage stability
FTD: Freeze-thaw damage
 G_f : Fracture Energy
LDPE: Low density polyethylene
RPR: Retained penetration ratio
SPI: Softening point increment
VAI: Viscosity aging index
 G^* : Complex shear modulus
LCA: Life Cycle Assessments
XRD: X-Ray Diffraction

Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Groundwater-Induced Changes in the Mechanical Response of a Shallow Strip Foundation in Sand

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Keywords

Groundwater table,

Strip footing,

Settlement,

Effective stress,

PLAXIS 2D.

Abstract

This study numerically investigates the effect of groundwater table (GWT) depth on the settlement behaviour, effective vertical stress distribution, and plastic zone development of a shallow strip footing using PLAXIS 2D. A homogeneous sand deposit was modelled under plane-strain conditions using the drained Mohr-Coulomb model. A uniformly distributed pressure of 150 kPa was applied to a 2 m wide surface strip footing. The footing geometry, loading conditions, and mechanical soil parameters were kept constant, while the groundwater table depth was varied from $D_w/B=0$ to $D_w/B=3.0$. The results were compared with a dry reference condition. When the GWT was located at the footing level, the maximum settlement was 28.4 mm, compared with 25.3 mm under dry conditions, corresponding to an increase of about 12.3%. Settlement decreased gradually as the GWT became deeper, while the water-table correction factor C_w approached unity. The influence of groundwater became limited beyond a depth of approximately two footing widths below the footing base. Effective stress results showed that the effect of groundwater became more pronounced with depth. At $z/B=3$, the effective vertical stress under the surface GWT condition was about 37% lower than that under dry conditions. Plastic-point distributions also indicated a wider plastic zone for the surface GWT condition. Overall, the results show that GWT position is an important parameter in assessing the service response of shallow foundations.

1. Introduction

Shallow foundations are among the most widely used foundation systems for transferring structural loads to the ground. Their safe design requires consideration of both ultimate bearing capacity and settlement under service conditions. In granular soils, footing settlement can be strongly influenced by the applied load, footing geometry, soil stiffness and strength, as well as the groundwater table. A rise in the groundwater table due to seasonal variations, heavy rainfall, flooding, or local hydrological conditions can alter the stress state beneath the foundation and lead to additional settlements that may not have been considered during design [1, 2]. Shahriar et al. (2013) [1] showed that, in granular soils, additional settlement increases as the groundwater table approaches the footing base and that the water-table correction factor varies significantly with groundwater depth.

The influence of groundwater on foundation behaviour is closely related to the effective stress principle. As the groundwater table rises, pore-water pressure increases, reducing the effective stresses carried by the soil skeleton and, consequently, the effective confining pressure. This change can directly affect the deformation and strength characteristics of granular soils. Morgan et al. (2010) [3] investigated the additional settlement caused by groundwater rise through laboratory tests and showed that the effect was generally more pronounced in loose sands than in dense sands. The same study also highlighted the importance of changes in Young's modulus and effective confining stress associated with saturation in explaining the additional settlement mechanism.

The effect of groundwater level on shallow foundation performance has been widely investigated using both experimental and numerical approaches. Shahriar et al. (2015) [2] experimentally examined water-table correction factors for different footing geometries and sand densities. Their results showed that C_w (the ratio of settlement under a given groundwater condition to the settlement in dry sand) increases as the groundwater table approaches the footing level. They also indicated that C_w approaches unity when the groundwater table falls below the settlement influence zone. Park et al. (2019) [4] investigated rising and falling groundwater cycles through controlled model tests. They reported that the groundwater influence depth may vary approximately between $0.3B$ and $1.5B$, and that settlements during groundwater rise can be greater than those observed during the falling stage.

Numerical studies have also demonstrated the importance of groundwater conditions in foundation behaviour. Bouaicha et al. (2018) [5] investigated the effect of groundwater level on the bearing capacity of strip footings on sand using two-dimensional numerical analyses and reported that changes in pore-water pressure can adversely affect soil strength. Cascone et al. (2021) [6] developed a groundwater-dependent bearing capacity correction approach for strip footings under plane-strain conditions. They also evaluated the size and shape of the plastic zone developing beneath the footing using finite element analyses. More recently, Abmdas and Al-Awamy (2025) [7] investigated the effect of GWT for different footing sizes and embedment depths using PLAXIS 3D and showed that shallow groundwater can cause significant reductions in bearing capacity.

Groundwater-related problems are not limited to bearing capacity. Kim et al. (2017) [8] investigated the effects of rainfall infiltration and different groundwater table positions on the settlement behaviour of shallow foundations using sequential flow and deformation analyses. Their results showed that groundwater table position is an important factor influencing foundation deformation. However, much of the existing literature has focused on ultimate bearing capacity, water-table correction factors, or saturation-dependent changes in soil stiffness. Therefore, evaluating settlement, depth-dependent effective vertical stress distribution, and plastic behaviour together under a constant service load, while systematically varying only the groundwater table position, provides a more comprehensive interpretation of the mechanical effects of groundwater.

In this study, a surface strip footing resting on homogeneous sand was modelled using PLAXIS 2D. Groundwater table conditions of $D_w/B=0, 0.5, 1.0, 1.5, 2.0$, and 3.0 were compared with a dry reference condition. The main contribution of the study is the isolation of groundwater table effects by keeping the footing geometry, applied load, and mechanical soil parameters constant. The response was evaluated not only in terms of maximum settlement, but also through C_v , depth-dependent effective vertical stress profiles, and the development of the plastic zone. This approach aims to identify the depth range over which groundwater has a meaningful influence on the service response of the investigated soil-foundation system and to determine when its effect becomes negligible at greater groundwater depths.

2. Materials and Methods

2.1. Numerical Model

The effect of groundwater table depth on the behaviour of a surface strip footing was investigated using the two-dimensional finite element method in PLAXIS 2D. The model was defined under plane-strain conditions, as appropriate for strip footing problems, and 15-node triangular elements were used. Similar modelling approaches have also been adopted by Hakro et al. (2022) [9] and Cicek et al. (2014) [10].

As shown in Figure 1, the soil domain was 30 m wide and 15 m deep. A strip footing with a width of $B=2$ m was placed at the ground surface ($D_f=0$) at the centre of the model. Horizontal displacement was prevented along both lateral boundaries, whereas the base of the model was fixed against displacement in both directions. The model dimensions were selected to minimise the influence of the boundaries on the deformation and plastic zone developing around the footing. These boundary conditions are consistent with similar PLAXIS-based strip footing analyses. The adopted footing width ($B=2$ m) and applied pressure ($q=150$ kPa) were selected as representative baseline values, consistent with the same width-load combination previously employed in a numerical investigation of shallow footing response by Kholdebarin et al. (2016) [11]. The uniformly distributed vertical pressure was applied as a line load over the footing area and was kept constant in all analyses. Hakro et al. (2022) [9] also represented distributed footing pressure as a line load in PLAXIS.

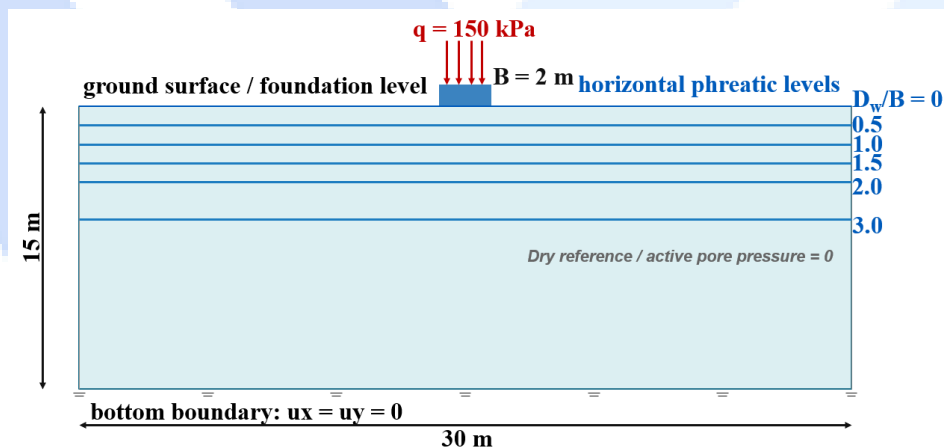


Figure 1. Numerical model geometry and normalised groundwater table levels.

2.2. Soil Properties and Analysis Conditions

The soil profile was modelled as homogeneous sand, and its behaviour was represented using the drained Mohr-Coulomb model. The mechanical parameters were selected in accordance with the values used by Cicek et al. (2014) [10] in their finite element analysis of a strip footing on sand (Table 1). The saturated and unsaturated unit weights and Poisson's ratio were adopted as typical values for sand used in PLAXIS.

The Mohr-Coulomb model was adopted since the study focused on the relative effect of groundwater table position while keeping the soil parameters constant in all analyses. Its simplicity and common use in PLAXIS-based shallow foundation analyses make it suitable for this comparative assessment [9]. However, its stress-independent stiffness may limit the accuracy of absolute settlement predictions compared with more advanced models such as Hardening Soil. A separate mesh sensitivity study was not performed; however, an identical 15-node finite-element discretisation was maintained in all analyses to provide a consistent basis for comparison.

The same footing geometry, loading condition, finite element mesh, and mechanical soil properties were maintained in all models, while only the groundwater table depth was varied. This allowed the results of the different analyses to be evaluated directly in relation to GWT position. The initial stresses were generated using the K_0 procedure, and the footing load was activated in the subsequent plastic calculation phase.

Table 1. Soil parameters used in the numerical analyses.

Parameters	Value
Soil model	Mohr-Coulomb
Drainage type	Drained
γ_{unsat}	17 kN/m ³
γ_{sat}	20 kN/m ³
E'_{ref}	25000 kPa
ν	0.3
c'	1
ϕ'	38°
ψ	8°

2.3. Groundwater Table Scenarios and Evaluation of Results

The groundwater table was defined as a horizontal hydrostatic phreatic level. Here, D_w represents the vertical distance between the footing base and the groundwater table. Since the footing was placed at the ground surface ($D_f=0$), D_w is also equal to the groundwater depth measured from the ground surface.

The groundwater table depth was normalised by the footing width, B , and values of $D_w/B=0, 0.5, 1.0, 1.5, 2.0$, and 3.0 were considered together with a dry reference condition. For $B=2$ m, these values correspond to groundwater depths of 0, 1, 2, 3, 4, and 6 m, respectively. Normalising groundwater depth by the footing width has also been adopted in previous studies on groundwater-induced settlement. Shahriar et al. (2013) [1] related the settlement correction factor to groundwater depth, while Park et al. (2019) [4] reported that the groundwater influence zone may vary over a depth range defined relative to the footing width, B .

Each GWT condition was analysed independently. Footing settlement was evaluated in terms of the maximum vertical displacement, u_y . Following the definition adopted by Shahriar et al. (2013, 2015) [1, 2], the water-table correction factor was calculated as $C_w=S_{\text{GWT}}/S_{\text{dry}}$. To examine the variation of effective vertical stress with depth, σ'_{yy} values were compared along the footing centreline at $z/B=1, 2$, and 3 . These locations correspond to coordinates of (15, 13), (15, 11), and (15, 9) m, respectively (Figure 2). Plastic-point distributions obtained from the plastic calculation phase were also examined. They were used as a qualitative indicator of changes in the extent of the plastic zone under the same 150 kPa load rather than as an independent performance measure.

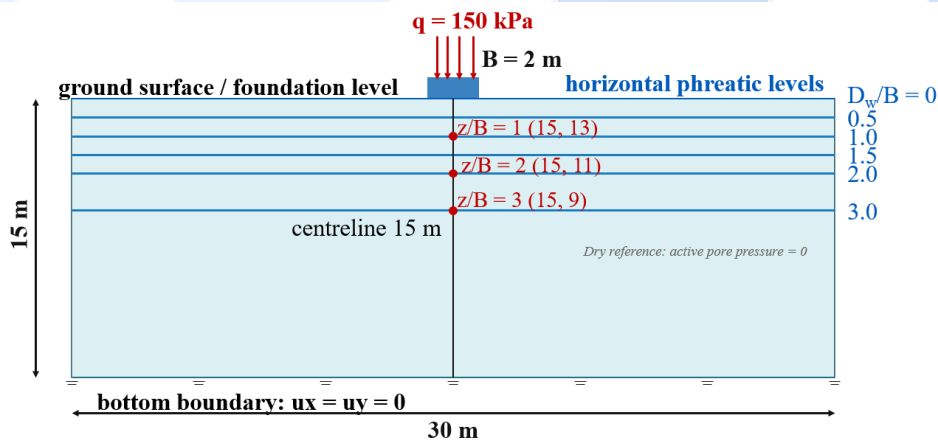


Figure 2. Normalized depths used to evaluate effective vertical stress along the footing centerline.

3. Results and Discussion

3.1. Effect of Groundwater Table Depth on Footing Settlement

Table 2 presents the maximum footing settlements calculated for the different groundwater table levels together with the corresponding water-table correction factors, C_w . The largest settlement, 28.4 mm, was obtained for $D_w/B=0$, where the groundwater table was located at the ground surface and footing base level. As the groundwater table became deeper, the maximum settlement decreased gradually to 26.9, 26.0, 25.6, 25.4, and 25.3 mm for $D_w/B=0.5, 1.0, 1.5, 2.0$, and 3.0 , respectively. The maximum settlement under the dry reference condition was also 25.3 mm. Therefore, locating the groundwater table at the footing level increased the maximum settlement by approximately 12.3% compared with the dry condition.

This behaviour is generally consistent with previous studies showing that footing settlement increases as the groundwater table approaches the foundation. Shahriar et al. (2013) [1] showed that C_w increases as the groundwater table rises and that the rate of additional settlement becomes more pronounced when the GWT approaches the footing level. They also reported that a rise in the groundwater table from 1.0B to 0.5B produces greater additional settlement than an equal rise from 2.0B to 1.5B. Similarly, Kim et al. (2017) [8] compared GWT positions at 1B and 2B and reported greater footing settlement for the shallower groundwater condition.

Table 2. Maximum footing settlement and water-table correction factor for different groundwater conditions.

Condition	D_w/B	s_{max} (mm)	$C_w = s_{GWT}/s_{dry}$
GWT = 0 m	0	28.4	1.123
GWT = 1 m	0.5	26.9	1.065
GWT = 2 m	1	26.0	1.029
GWT = 3 m	1.5	25.6	1.012
GWT = 4 m	2	25.4	1.004
GWT = 6 m	3	25.3	1.000
Dry	-	25.3	1.000

C_w decreased from 1.123 at $D_w/B=0$ to 1.000 at $D_w/B=3.0$, showing a gradual reduction in groundwater influence with depth. This trend is consistent with Shahriar et al. (2015) [2], who reported a more pronounced increase in the water-table correction factor as the groundwater table approaches the footing base. They also stated that $C_w=1$ when the groundwater table lies below the settlement influence zone, while C_w reaches its maximum value when the GWT rises to the footing level.

For the analysed soil-foundation system, C_w reached 1.004 at $D_w/B=2.0$, while the settlement at $D_w/B=3.0$ was the same as that obtained under dry conditions. This indicates that the influence of groundwater became very limited beyond a depth of about two footing widths. This result is generally consistent with the groundwater influence zone of approximately $0.3B$ – $1.5B$ reported by Park et al. (2019) [4], although previous studies have also shown that groundwater effects may extend to greater depths. Shahriar et al. (2015) [2], for example, reported additional settlements at depths of several footing widths under different sand densities and test conditions. This suggests that the groundwater influence depth is not a fixed geometric limit, but depends on soil properties and the adopted modelling approach. The obtained C_w values followed the expected groundwater-dependent trend reported in previous studies. Settlement increased as the groundwater table approached the footing level, while C_w approached unity as the GWT became deeper.

3.2. Effect of Groundwater Table Depth on Effective Vertical Stress Distribution

The effect of different groundwater table conditions on the effective vertical stress distribution beneath the footing was examined. To evaluate the variation of effective vertical stress with depth, the normalised depths $z/B=1, 2$, and 3 were considered. As shown in Table 3, the effective vertical stresses at $z/B=1$ varied within a narrow range of approximately 119.4–122.4 kPa for all GWT conditions. The differences between the GWT cases became more noticeable with increasing depth. At $z/B=2$, the effective vertical stress was 88.4 kPa when the groundwater table was at the ground surface, compared with 106.8 kPa under dry conditions. This corresponds to a reduction of approximately 17.2%. At $z/B=3$, the effective stress was 80.3 kPa for GWT = 0 m and 127.5 kPa for the dry condition, giving a difference of approximately 37%.

Table 3. Effective vertical stress magnitudes along the footing centreline for different groundwater conditions.

Condition	D_w/B	σ'_{yy} at $z/B=1$ (kPa)	σ'_{yy} at $z/B=2$ (kPa)	σ'_{yy} at $z/B=3$ (kPa)
GWT = 0 m	0	119.4	88.4	80.3
GWT = 1 m	0.5	122.4	88.1	88.5
GWT = 2 m	1	121.7	92.4	97.7
GWT = 3 m	1.5	121.9	99.2	105.7
GWT = 4 m	2	122.0	105.3	113.1
GWT = 6 m	3	122.0	106.7	126.5
Dry	-	122.0	106.8	127.5

Figure 3 shows the variation of effective vertical stress with normalised depth along the footing centreline. For clarity, only the GWT=0 m, GWT=2 m, and dry conditions are shown. Near the footing, the stress profiles are very similar, indicating that the applied footing pressure dominates the stress response in this region. At depths greater than approximately $z/B=1$, the profiles begin to separate. The effective vertical stress remains lower for GWT=0 m, while the profile approaches the dry condition as the groundwater table becomes deeper. Under the dry condition and conditions with a deeper GWT, the initial decrease and subsequent increase in effective vertical stress can be explained by two competing effects. The stress contribution from the applied footing load decreases with depth, while the geostatic effective stress due to soil self-weight increases with depth. Cicek et al. (2014) [10] showed that the vertical stress distribution beneath strip footings varies with both the applied load and normalised depth. Their measured stress profiles also indicate that the additional vertical stress generated in the soil by the footing load decreases with increasing normalised depth.

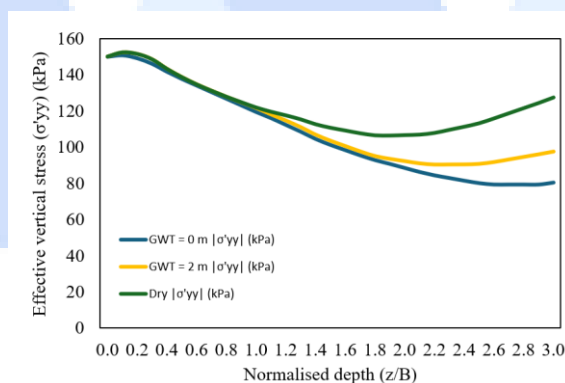


Figure 3. Variation of effective vertical stress with normalised depth along the footing centreline for selected groundwater conditions.

The effective vertical stress contours presented in Figure 4 confirms the same behaviour. Under dry conditions, higher effective stress magnitudes are observed throughout the soil beneath the footing. When the groundwater table is located at the ground surface, the effective stresses are noticeably lower. Thus, the tabulated values, centreline stress profiles, and contour plots provide consistent and complementary results.

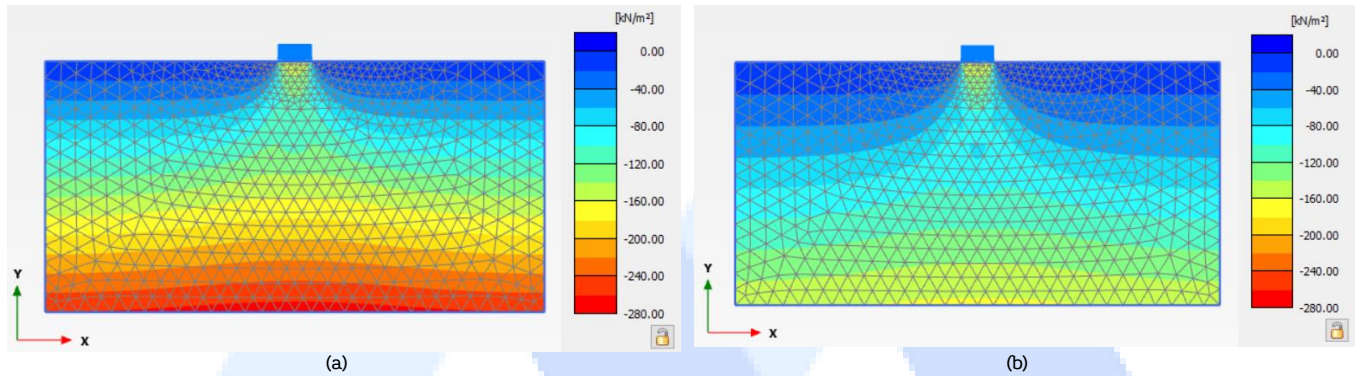


Figure 4. Effective vertical stress distributions under (a) dry and (b) groundwater-at-surface conditions. The same legend range was used to facilitate direct comparison between the two cases.

This behaviour can be explained by the effective stress principle. As the groundwater table rises, hydrostatic pore-water pressure increases in the saturated zone, which reduces the effective stress carried by the soil skeleton. Shahriar et al. (2013) [1] reported that when granular soil becomes saturated, effective stresses and the associated confining stresses decrease. Similarly, Cascone et al. (2021) [6] stated that pore-water conditions and groundwater table depth directly affect the effective stress state of the soil, and that submergence beneath the footing reduces effective stresses.

3.3. Effect of Groundwater Table Position on The Plastic Zone

To evaluate the effect of groundwater table position on yielding beneath the footing, plastic-point distributions from the plastic calculation phase were examined. For a clearer comparison, the dry reference condition and the GWT=0 m condition are presented together in Figure 5. The plastic points were treated as a qualitative indicator of the extent of the soil zones reaching the yield condition under the same footing pressure.

Figure 5 shows that plasticisation is concentrated within a more limited zone under the dry condition (Figure 5a), whereas the GWT=0 m condition (Figure 5b) produces a wider and deeper distribution of plastic points beneath the footing. In both cases, plasticisation develops mainly below the footing base and extends downwards and laterally. The affected soil zone is more pronounced when the groundwater table is at the ground surface. This observation is also consistent with the higher footing settlement obtained for GWT=0 m under the same applied load. Plastic-point distributions have also been used in the literature to visualise the development of yielding beneath shallow foundations. Emren and Tuncay (2021) [12], for example, presented PLAXIS plastic-point distributions to examine the development of the failure mechanism beneath shallow footings.

This behaviour can be interpreted together with the effective stress changes discussed in the previous section. As the groundwater table rises, the increase in pore-water pressure reduces the effective stress and, consequently, the effective confining pressure in the soil. According to the Mohr-Coulomb failure criterion, shear strength depends on effective normal stress. Therefore, under the same footing load, lower effective stress may cause a greater number of stress points to approach or reach the yield condition. Cascone et al. (2021) [6] also reported that saturation of the soil beneath the footing reduces effective stresses and the associated shear strength. This mechanism explains the wider plastic zone and the higher settlement observed for the GWT=0 m condition.

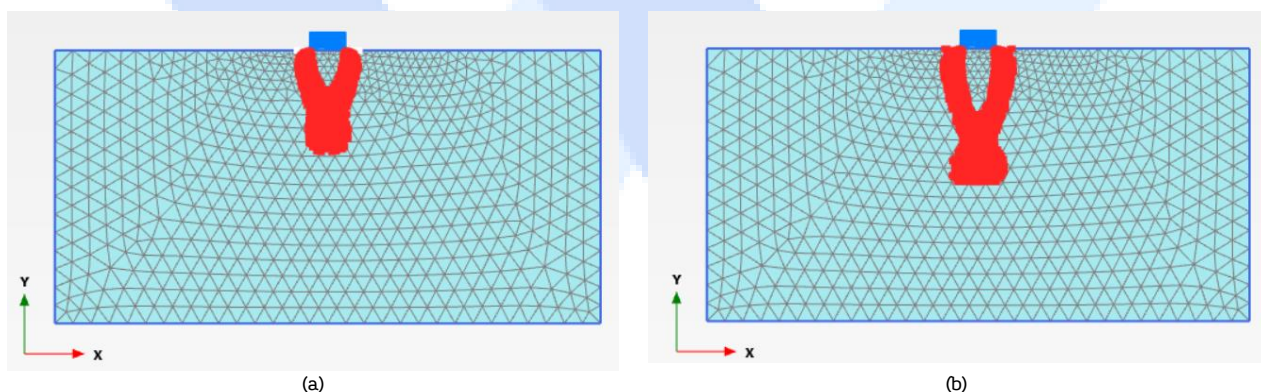


Figure 5. Plastic-point distributions obtained at the end of the plastic calculation phase for (a) dry and (b) GWT=0 m conditions.

4. Conclusions

This study investigated the effect of groundwater table depth on the settlement behaviour, effective vertical stress distribution, and plastic zone beneath a surface strip footing using PLAXIS 2D analyses. The footing geometry, applied load, and mechanical soil parameters were kept constant, while only the groundwater table position was varied. The main findings are summarized below:

- When the groundwater table was located at the footing level $D_w/B=0$, the maximum settlement was 28.4 mm, compared with 25.3 mm under dry conditions. This corresponds to an increase of approximately 12.3%. As the GWT became deeper, settlement gradually decreased and C_w approached unity from 1.123. The groundwater effect became very limited beyond $D_w/B=2.0$, and the settlement at $D_w/B=3.0$ was the same as that obtained under dry conditions.
- The effect of GWT on effective vertical stress was found to depend on depth. At $z/B=1$, the differences between the GWT conditions were limited, whereas at $z/B=3$, the effective stress was 80.3 kPa for GWT=0 m and 127.5 kPa under dry conditions. The stress profiles and contours showed that effective stress decreased as the groundwater table rose, while the response approached the dry condition as the water table became deeper.
- At the end of the plastic calculation phase, the plastic zone under the GWT=0 m condition was observed to extend over a wider and deeper region than under dry conditions. This response is consistent with the lower effective stresses and higher footing settlement obtained for the surface groundwater condition.

Overall, the results show that groundwater table position affects footing settlement, effective stress distribution, and plastic behaviour. For the investigated soil-foundation system, the influence of groundwater became progressively less pronounced as the GWT became deeper. For a denser and stiffer sand, lower absolute settlements and a less pronounced deformation response to groundwater rise would be expected, while the general depth-dependent effective stress trend would likely remain similar. The present study provides a controlled parametric assessment of this behaviour. Future studies should consider different sand densities, footing widths, load levels, and more advanced soil models to examine the response under a wider range of conditions.

Nomenclature

B: Footing width

C_w : Water-table settlement correction factor

D_f : Footing embedment depth

D_w : Vertical distance between the footing base and groundwater table

K_0 : Coefficient of earth pressure at rest

s_{GWT} : Footing settlement under a given groundwater condition

s_{dry} : Footing settlement under dry conditions

z: Depth below the footing base

D_w/B : Normalised groundwater table depth

z/B : Normalised depth below the footing base

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Numerical Analysis and Production of a Mobile Industrial Secondary Structure Designed with Glass-Fiber Reinforced Plastic Profiles: A Case Study

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Keywords

Glass Fiber-reinforced plastic (GFRP),

Numerical modeling,

Mobile secondary structure,

Special profile detailing,,

Case study.

Abstract

This study presents the engineering processes of a mobile secondary structure designed for use in industrial facilities using Glass-Fiber Reinforced Plastic (GFRP) composite profiles as a case study. Within the scope of the study, the material's stiffness and strength parameters were determined by analyzing comprehensive mechanical test results, including "dog-bone" axial tensile tests. Based on the obtained material data, a three-dimensional (3D) numerical model of the structure was created using SAP2000 finite element software, and structural analyses were performed. During the design process, unique connection and detailing solutions were developed, considering the geometric constraints and detailing challenges associated with profiles specifically manufactured for composite materials. The mobile framework system, designed to suit industrial site conditions, was optimized to safely support valves, piping, and control elements, and was subsequently manufactured into a final product. The system was produced for temporary operation under the manufacturer's commitment to cover any potential operational expenses arising from unexpected conditions. Since a unified and directly applicable national design framework for the structural use of pultruded GFRP profiles in the investigated application is not readily available, the study offers a holistic overview of the practical limitations and solution approaches encountered ranging from the numerical modeling of GFRP profiles to final manufacturing in the context of industrial structures.

1. Introduction

The design of principal lateral load systems is only one aspect of structural safety in industrial facilities; other secondary structural elements that perform different roles within the facility must also be reliably designed. Even though these components are frequently smaller than core load-bearing systems, they are essential for process safety, equipment operational continuity, personnel safety, and the viability of maintenance tasks. Material selection for these structures necessitates the simultaneous assessment of multiple performance criteria beyond mere load-bearing capacity, such as corrosion resistance, weight, fire performance, and electrical properties, especially in industry with harsh conditions like chemical, oil and gas, and water treatment plants [10, 11].

Although steel profiles are traditionally used in industrial facilities [4, 12-13], issues such as high density, continuous protective coating and maintenance requirements, and corrosion issues have enhanced needs for alternative materials. Glass Fiber Reinforced Plastic (GFRP) composite profiles having properties such as high specific strength, corrosion vulnerability have underscored the need for alternative structural materials, particularly for modular, mobile and secondary industrial facilities. Previous research proved that the mechanical behavior of Glass-Fiber Reinforced Plastic GFRP composites depending on fiber orientation, loading conditions, and material composition [14]. El-Assal and Khashaba (2007) showed that the mechanical and fatigue behaviors of Glass-Fiber Reinforced Plastic (GFRP) composites can vary under different loading conditions, while AlAjarmeh et al. (2019) demonstrated that the performance of GFRP-based elements in structural applications depends on design parameters and material properties, so that using of GFRP can provide meaningful benefits, especially for corrosive industrial environments [1, 6].

For usage of GFRP profiles in real structures, it is compulsory to precisely determine the material's mechanical parameters and justify them with suitable experiments. Specifically in pultruded profiles, the predominant alignment of fibers along the profile axis may cause in varying stiffness and strength values for the transverse and axial directions. Because of this, GFRP profiles require a different design strategy than traditional isotropic steel profiles. Tensile tests performed in compliance with the TS EN ISO 527-1/4 standard on the FIBERR-brand GFRP profile utilized in this investigation revealed significant directional dependency [8]. However, real industrial systems, profiles do not operate only under uniform axial loads; they can be subjected to stress concentrations due to joints, cross-sectional changes, holes, connections, and equipment attachments. This case causes a crucial design challenge, especially when the direction-dependent behavior of composite materials is under consideration. Applied stress concentration level has a great effect on fatigue life according to study in which the fatigue behavior of GFRP composites using experimental, analytical, and finite element approaches were investigated done by Ferdous et al. (2020) [7]. Moreover, Allamraju (2018), highlighted the importance of diminishing geometric features that cause stress concentrations for GFRP composites [2].

Along with mechanical characteristics, reaction-to-fire properties are an important design criterion when using GFRP profiles for industrial applications. According to the ASTM E84-16 standard [9], the profile material's surface burning characteristics were evaluated experimentally, yielding a Spread Index of 5 and a Smoke Developed Index of 900. Additionally, the profile samples satisfied the requirements for the B S2 d0 class under TS EN 13501-1, according to reaction-to-fire tests carried out in compliance with TS EN 13823 and TS EN ISO 11925-2 standards [18-21]. The electrical insulation performance of GFRP profiles is another feature that supports their usage in particular industrial applications. The electrical characteristics of structural elements can be used as an extra safety criterion in buildings that contain electrical equipment, power systems, or conductive components. For this research, a high-voltage dielectric breakdown test was carried out on the FIBERR pultrusion profile in accordance with the TS EN 60243-1 standard [17].

When these features are taken into consideration, GFRP profiles should be viewed as a composite structural system with specific design requirements rather than just as a material substitute in industrial applications. Load transmission techniques, fiber orientation, profile shape, and connection specifics are not independent variables. The development of mechanical connections and the geometric details around them is required since the welded connections typically employed in steel buildings cannot be directly transferred to GFRP profiles. Consequently, it is crucial to assess connection and detailing solutions in conjunction with system behavior and material characterization when constructing secondary structures utilizing composite profiles.

The transportable secondary structural system used in industrial facilities that is made from GFRP composite profiles is the subject of this study. This study aims to investigate the engineering technique, extending from the experimental verification of GFRP material properties to the design and manufacture of an actual industrial product. As a result, the results of experimental characterization were used as design inputs; SAP2000 [5] software was used to generate a three-dimensional numerical model of the system, and structural analyses were carried out. The material-level mechanical behavior of GFRP composites has been thoroughly studied in previous research; however, the integrated structural design, connection detailing, seismic assessment, and physical realization of mobile industrial secondary systems made from pultruded GFRP profiles have received relatively little attention. The planned transportable system was eventually produced as a physical product after being assessed for its capacity to securely support machinery in industrial facilities, including pipes, valves, and control components. Therefore, the study proposes an overall, application-oriented engineering approach that goes beyond solely examining the material properties of GFRP profiles, integrating the phases of experimental characterization, numerical modeling, connection design, and final manufacturing.

2. Materials and Experimental Methodology

2.1. Tensile Testing of GFRP Specimen

In this research, to ascertain the mechanical behavior of reinforced plastic composite specimens in the transverse and axial directions, tensile tests were performed in compliance with the TS EN ISO 527-1/4 standard. [8] and shared. The results of dog bone test are presented in Table 1, and the details of dog-bone test specimens are presented in Fig. 1.

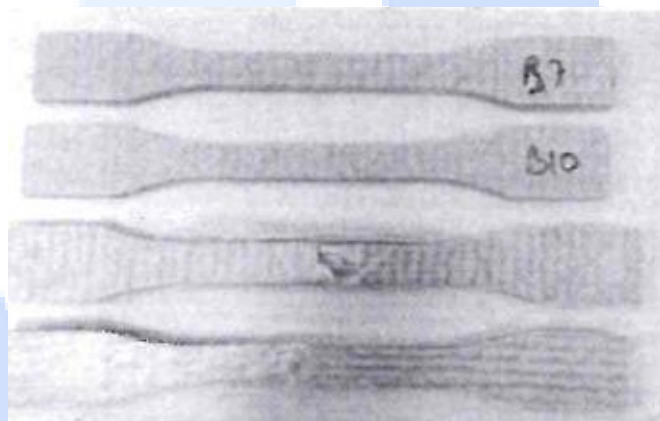


Figure 1. Details of dog-bone test specimens [8]

Table 1. Results of dog bone test [8]

Direction	Specimen	Elasticity Modulus (GPa)	Stress (MPa)	Elongation (%)	Thickness (mm)	Width (mm)
Axial	1	28.5	374.0	1.4	5.26	10.15
	2	26.9	361.0	1.2	5.23	10.13
	3	27.3	374.0	1.5	5.19	10.20
	4	26.4	372.0	1.6	5.19	10.13
	5	26.2	372.0	1.5	5.20	10.13
	Average	27.06	370.6	1.4	5.21	10.15
	Standard deviation	0.9	5.5	0.2		
Transverse	1	9.09	79.8	1.4	5.15	10.14
	2	9.42	78.4	1.2	5.18	10.18
	3	9.07	78.8	1.0	5.17	10.17
	4	8.54	77.3	1.4	5.10	10.15
	5	8.04	78.8	1.3	5.20	10.20
	Average	8.83	78.6	1.3	5.16	10.17
	Standard deviation	0.5	0.9	0.2		

According to table 1, The five specimens' modulus of elasticity values in the axial direction varied from 26.2 to 28.5 GPa, with an average of 27.06 GPa. On the other hand, the transverse modulus of elasticity values averaged 8.83 GPa and varied from 8.04 to 9.42 GPa. These findings show that the GFRP material has unique mechanical behavior that is both orthotropic and anisotropic. The axial stiffness is around three times greater than the transverse stiffness because of the fibers' alignment along the principal loading axis. Tensile stress has a similar tendency. It was found that the average tensile stress was 78.6 MPa in the transverse direction and 370.6 MPa in the axial direction. As a result, the axial tensile strength is roughly 4.7 times greater than the transverse strength. This illustrates how fiber orientation significantly affects GFRP composites' load-bearing capability and emphasizes how crucial it is to take loading directions into account when designing materials. The deformation capacity beyond the elastic domain is represented by elongation values, which are roughly 1.3–1.4% in both directions. The material is primarily intended to offer great strength along the fiber axis, as seen by the high tensile strength measured in the axial direction. Additionally, low standard deviation values show that the experimental results exhibit strong reproducibility and are closely grouped.

2.2. Mechanical and Physical Characterization Standards

The Certificate of Conformity issued by the TSE (Turkish Standards Institute) certifies that the mechanical strength values such as tensile, flexural, and shear strength of pultruded reinforced plastic composite profiles from the sources of TEZCOM TSE, RENCO TSE and PULTECH TSE have been verified through tests conducted in accordance with the TS EN 13706-2 standard [16] and shared. Table 2 shows the mechanical properties of GFRP. The three manufacturer/technical sources' figures are generally comparable. The manufacturer data show that the axial tensile strength is 240 MPa and the transverse tensile strength is 50 MPa; the axial and transverse moduli of elasticity are 23 GPa and 7 GPa, respectively. Shear strength is stated to be 25 MPa, while axial and transverse flexural strengths are reported to be 240 MPa and 100 MPa, respectively. Additionally, the axial punching/shear load-bearing capacity is 150 MPa, whereas the transverse direction is 70 MPa. The mechanical properties of the GFRP material utilized in the study are equivalent to those found in literature and manufacturer technical specifications, as evidenced by the fact that manufacturer data are same or extremely similar across three sources. However, there is a noticeable difference when compared to the experimental data in Table 1, especially regarding tensile strength. [16].

Table 2. Mechanical Properties of GFRP

Explanation		TEZCOM TSE	RENCO TSE	PULTECH TSE
Tensile Force				
Axial	MPa	240	240	240
Transverse	MPa	50	50	50
Elasticity Modulus				
Axial	GPa	23	23	23
Transverse	GPa	7	7	7
Flexural Strength				
Axial	MPa	240	240	240
Transverse	MPa	100	100	100
Shear Strength	MPa	25	25	25
Punching shear bearing capacity				
Axial	MPa	150	150	
Transverse	MPa	70	70	

According to Tables 1 and 2, GFRP material exhibits distinct directional mechanical behavior. The average modulus of elasticity and tensile strength in the axial direction were found to be 27.06 GPa and 370.6 MPa, respectively, based on dog-bone tensile test findings; in the transverse direction, these values were found to be 8.83 GPa and 78.6 MPa. As a result, the tensile strength and axial modulus of elasticity are around 4.7 and 3.1 times the values in the transverse direction, respectively. This demonstrates how fiber orientation has a significant impact on the mechanical performance of GFRP composites. Both the modulus of elasticity and tensile strength were higher in the experimental results when compared to the figures given by the manufacturer in Table 2. The experimental tensile strength was 370.6 MPa in the axial direction compared to the manufacturer's 240 MPa, and it was 78.6 MPa in the transverse direction compared to the manufacturer's 50 MPa. In the same way, the experimental elasticity modules were approximately 18% and 26% higher than the manufacturer's values, respectively. Variations in the material's fiber volume fraction, fiber orientation, manufacturing conditions, resin characteristics, and testing conditions are the reasons for these disparities. However, the test findings' low standard deviation values show that the mechanical properties are highly repeatable and show that the experimental data accurately depicts the mechanical behavior of the GFRP material utilized in the investigation.

In addition, to ascertain the surface fire behavior and smoke production characteristics of the FRP square tube/profile material owned by the business FIBERR, the ASTM E84-16 test (surface burning characteristics) was performed. The material passed the test with a Flame Spread Index of 5 and a Smoke Developed Index of 900 [9]. Glass fiber-reinforced composite (GFRP) box profile samples were tested for fire resistance and reaction to fire in compliance with TS EN ISO 11925-2 (ignitability test with a single flame source) and TS EN 13823 (reaction to fire/SBI test with a single burning item) [18-21]. The glass fiber-reinforced profile samples were determined to satisfy the requirements for the B S2 d0 reaction-to-fire class as specified in the TS EN 13501-1 standard based on the fire tests conducted [18-21]. According to the TS EN ISO 11925-2 standard, a glass fiber-reinforced composite (GFRP) box profile sample was subjected to a direct flame exposure and ignitability (response to fire) test utilizing a single flame source. The test findings demonstrated that even though the samples burned during the 30-second flame application, the product satisfied the Class B requirements of the applicable standard since the molten droplets did not ignite the filter paper, and the flame spread did not exceed the 150 mm limit [18-21]. In this document, a FIBERR-branded pultrusion profile sample was subjected to a High-Voltage Dielectric Breakdown Test in compliance with TS EN 60243-1 standards and a Surface Electrical Resistance Determination Test in compliance with IEC 60093 standards. The tests yielded a breakdown voltage of 44 kV and a surface electrical resistance of $10^{12} \Omega$ for the pultrusion profile sample [17].

3. Case Study

Examples of previously executed GFRP applications involving pipe supports, equipment supports, cable trays, ladders, cat walks, handrails, and mobile frame systems are presented in Fig. 2. The design and manufacturing of secondary mini mobile structures used as pipe support system was selected as a case study for this work. As GFRP material, 50*5 angle and 50*50*5 or 2.5 box profiles were used. Total height was 1.7 m; planar dimensions were 0.54 - 0.57 m.



Figure 2. Details of example GFRP applications

Peak ground acceleration (PGA) is 0.681 g. Since elastic spectrum was used, structural behavior coefficient R was taken as 1 and building importance factor was assumed as 1. Soil class was ZE. SDS and SD1 values were taken 1.329 and 1.039 in accordance with Türkiye Building Seismic Code [15]. Parameters SDS and SD1 represent the short-period and 1-second period mapped spectral design acceleration coefficients (dimensionless) respectively. The details of the isometric model are presented in Fig. 3. In the analysis, as a finite element software SAP 2000 [5] was used. Since there is no design regulation, steel design load combinations for Allowable Stress Design (ASD) were implemented. Equipment loads taken from mechanical design was added to model with suitable temperature loads (-18°C and $+30^{\circ}\text{C}$). No wind load was considered, because structure would be inside of a building. The analysis and design process were repeated until the calculated member capacity ratios remained below unity for the investigated structural configuration. All the material properties were defined with respect to test results by considering isotropy/anisotropy properties.

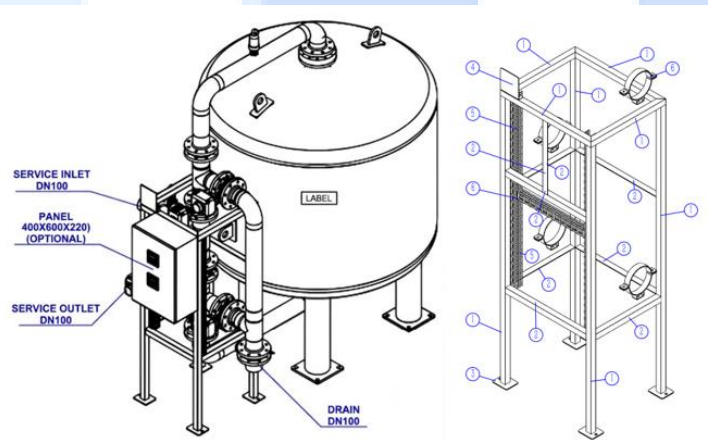


Figure 3. Details of model

4. Results and Discussion

The experimental results also showed the significant directional dependence of the GFRP material. The average axial tensile strength and modulus of elasticity of the profile were found to be 370.6 MPa and 27.06 GPa, respectively, whereas the equivalent values in the transverse direction were 78.6 MPa and 8.83 GPa. These findings show that the direction of loading and the profile's location within the structural system are important design considerations [8]. The experimentally obtained material properties were also higher than the corresponding manufacturer values presented in Table 2. This difference may be attributed to variations in fiber volume fraction, fiber orientation, manufacturing conditions, resin characteristics, and testing conditions. By considering this material data, structural analysis model created in SAP2000 [5] finite element software. From the analysis, in one direction, modal vibration period was 0.356 s, and in the other direction vibration period was 0.349 s. According to response spectrum analysis, base shear forces for both directions were 6.344 kN and 7.084 kN. The maximum displacements were 26 mm and 33 mm for two directions. Based on the lateral displacements, the corresponding drifts were calculated as 1.5% and 1.9%. While the maximum PMM ratio for a column was 0.643, whereas the maximum ratio for a beam was 0.393. Therefore, the calculated member capacity ratios remained below unity for the investigated structural configuration.



Figure 4. Connection details of GFRP profiles

In Fig. 4, connection details of GFRP profiles including base and brace connections are illustrated. This example connection detail gives rise to numerous assembly constraints. Drilling holes for bolted connections breaks the continuity of the fibers since GFRP profiles behave anisotropically. This results in a large concentration of stress around the holes and increases the risk of delamination or profile ripping at the connection point. Furthermore, while fitting the upright profile into the composite shoe utilized at the base connection, exact tolerance control is crucial. A completely fixed moment-resisting connection is not offered by the connection setup. The researched system's composite angle sections and connecting plates offer less rotational stiffness than welded steel connections. As a result, these connections might behave in a semi-rigid or near-pinned manner, which could lessen the frame's lateral stiffness and complicate assembly alignment.

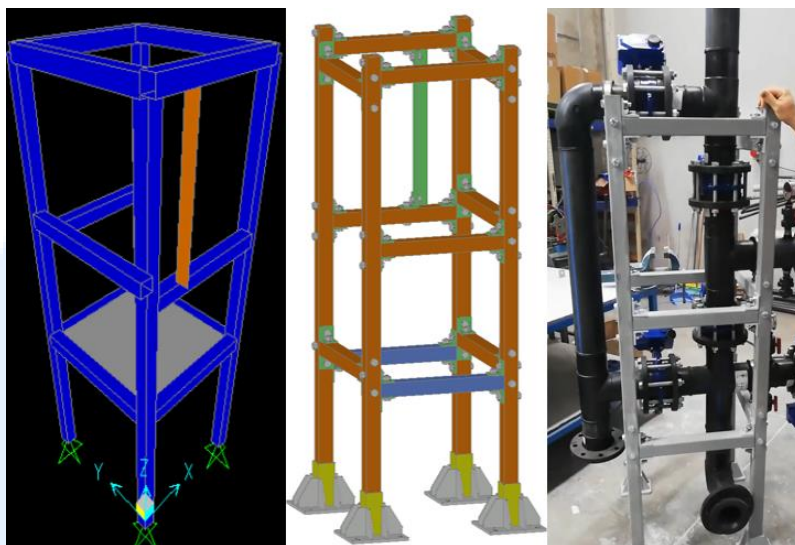


Figure 5. Finite element model [5], manufacturing model [3] and final physical appearance

Therefore, during the design phase, the connection and profile geometry should be considered together with the structural behavior and ease of assembly. Previous research has shown that stress concentration might negatively impact the fatigue performance of GFRP composites when compared to specimens with rounded features made of the same material [7]. This finding implies that for both structural behavior and ease of assembly, connection details and profile geometry should be considered at the design stage. The GFRP profiles' electrical characteristics and response to fire were examined in addition to their mechanical performance. The profile showed a Flame Spread Index of 5 and a Smoke Developed Index of 900 in the ASTM E84-16 test [9]. Additionally, the GFRP profile samples met TS EN 13501-1's standards for the B-S2 d0 reaction-to-fire class. In the direct flame exposure test carried out in accordance with TS EN ISO 11925-2, the samples burnt throughout the 30-second flame application; nevertheless, the flame spread did not above the 150 mm limit, and the molten droplets did not ignite the filter paper. These results offer more details about the substance under investigation's possible industrial use [18-21].

The electrical properties of the GFRP profile were also evaluated. The high-voltage dielectric breakdown test resulted in a breakdown voltage of 44 kV, while the surface electrical resistance was determined as $10^{12} \Omega$. These results indicate that the investigated GFRP profile possesses electrical insulation characteristics that may be advantageous in industrial environments where electrical isolation is required [17].

Finally, the model for manufacturing process and final physical appearance at site are shown in Fig. 5. Overall, the results indicate that the investigated GFRP profiles can be used to develop lightweight and transportable secondary structures for industrial applications when the directional material properties, connection details, structural stiffness, and relevant service conditions are properly considered.

5. Conclusion

This study assessed the computational and experimental performance of a mobile industrial secondary structure created with GFRP profiles. The outcomes can be summarized as follows:

- The directional behavior of the material with axial tensile strength and elastic modulus approximately 4.7 and 3.1 times higher than the corresponding transverse values respectively, must be considered during design since it was shown that the mechanical characteristics of GFRP profiles vary greatly depending on the fiber orientation.
- When material properties obtained from experiments were successfully used as inputs in numerical models, maximum PMM ratios of 0.643 and 0.393 for columns and beams, respectively, were found in the structural analysis, suggesting that the members' strength was sufficient.
- Inter-story drift ratios resulted in relatively high drift ratios of 1.5% to 1.9%, and maximum lateral displacements were reported to be between 26 and 32 mm. This highlighted that deformation control and lateral stiffness, rather than member strength alone, is a significant design consideration for the structures made of GFRP profiles.
- The large relative drift ratios have been shown to be caused by the interruption of fiber continuity at the GFRP profiles' connection zones and the practical difficulties in establishing a fully rigid (fixed) base connection. Consequently, connection stiffness needs to be properly considered, during the design and detailing stages of GFRP structures.
- Despite these limitations, thanks to the developed connection and production specifications such as electrical insulation and reaction-to-fire, the structure was successfully fabricated as a physical mobile industrial system, proving that GFRP profiles are suitable for use in lightweight, transportable secondary industrial structures.

Based on the results obtained from this study, the following future studies are recommended:

- To improve the system's lateral stiffness and lower relatively drift ratios, different base and anchoring connections should be developed and compared. Specifically, the experimental and computational assessment of stronger base connections is regarded as an important area of study.
- A parametric study of profile geometry, connection arrangement, and base connection details can be used to find the ideal design relationship between structural stiffness, weight, and load-bearing capacity. Additionally, it is important to investigate how long-term exposure to temperature, humidity, UV light, and chemicals affects GFRP profiles' mechanical performance.
- To ascertain the stiffness, deformation capacity, and failure behavior of GFRP, a comprehensive experimental study would be advantageous.

The material characteristics, numerical behavior, connection details, and manufacturability of a movable industrial secondary structure made using glass fiber-reinforced polymer (GFRP) profiles were all assessed in this study. A practical integrated methodology was demonstrated for transforming experimentally characterized pultruded GFRP profiles into a numerically assessed and physically manufactured mobile industrial secondary structure.

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Comparison of Lead-Rubber and Friction Pendulum Base Isolation Systems Based on Preliminary Design Criteria: A Case Study in Accordance with ASCE 7

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Keywords

Seismic base isolation design,

Friction pendulum system,

Lead-rubber bearing,

Vertical tension capacity,

Cavitation.

Abstract

In this study, a six-story reinforced concrete structure's seismic base-isolated design is examined as a case study in accordance with ASCE 7 standard. Preliminary designs for Lead Rubber Bearing (LRB) and Friction Pendulum System (FPS) isolators were carried out during the study's first phase. The selection of the isolation system was primarily governed by the tensile forces induced by overturning moments under seismic load combinations and the tensile capacity of elastomeric bearings up to the cavitation limit. The LRB system was therefore selected as the preferred isolation system for the investigated structure even though FPS systems are unable to transfer tensile stress because rubber-based LRB isolators provide a tensile strength advantage up to the cavitation limit (1.5–2.5 MPa). The determined LRB isolator parameters were used in the second phase to create a three-dimensional structural model using ETABS. Seven pairs of acceleration records scaled in compliance with ASCE 7 guidelines were then used for Nonlinear Time History Analyses (NTHA) and Response Spectrum Analyses. The first-mode period increased from 1.098 s for the fixed-base structure to 2.327 s after implementation of the isolation system, according to the dynamic analyses. The average peak displacement of the isolation layer was determined to be 276.2 mm under the Maximum Considered Earthquake (MCE) threshold, and it was confirmed that this value stayed within allowable bounds. Under the Design Basis Earthquake (DBE), the base shear coefficient acting on the superstructure has dropped to a level of 9%. It was observed that the axial tensile stresses developing in all isolators remained below the 1.0 MPa limit permitted by ASCE 7 and below the threshold for cavitation risk. The results indicate that the LRB system substantially reduces the seismic demands on the superstructure, ensuring it remains within the elastic behavior range, and effectively prevents the risk of cavitation.

1. Introduction

In areas where a large percentage of the current building stock presents seismic safety issues, mitigating earthquake-induced structural damage remains a major challenge in structural engineering. The significance of addressing seismic performance through both traditional strengthening techniques and cutting-edge protection systems that lessen the transmission of seismic effects to the structure is highlighted by studies evaluating the seismic risk of existing structures. The significance of assessing the current building stock in terms of seismic safety is further highlighted by a study by Ergen et al. (2025) on the seismic risk assessment of buildings in the Başakşehir district of Istanbul [5]. In this regard, seismic isolation systems seek to minimize energy, and acceleration demands placed on the building during an earthquake become an important design strategy, providing an alternative to conventional structural retrofit techniques. Base isolation lessens the superstructure's interaction with seismic ground motion by extending the structure's natural vibration period. This reduces key seismic response parameters, such as floor accelerations, base shear forces, and inter-story drifts.

Seismic isolation methods are used in a variety of engineering applications, such as bridges, vital infrastructure, energy facilities, and industrial plants, in addition to traditional building types like homes and offices. Through both experimental and analytical methods, Karayel et al. (2017) examined an alternative seismic isolation system that makes use of spring-tube braces and proved the system's suitability for a variety of structural types [7]. Similarly, the wide range of applications of friction pendulum isolators is demonstrated by their suitability for various engineering and infrastructure systems. The seismic behavior of machinery, pipelines, and other non-structural elements is particularly important for the overall functioning of industrial structures. Kılıç (2025) discussed the design of a multi-story steel industrial structure that housed several kinds of equipment, highlighting the importance of equipment effects on the design process and the seismic behavior of the structural system [10]. Therefore, it is crucial to consider behaviors like vertical forces and uplift effects that may occur at the isolation level in addition to the horizontal seismic demands when choosing seismic isolation systems.

This study's initial phase compared sliding-type Friction Pendulum System (FPS) isolation systems with elastomeric Lead Rubber Bearing (LRB) isolation systems. In FPS systems, sliding action over a spherical or curved sliding surface controls the structure's horizontal movement. The

system's effective period and seismic behavior are determined by several design parameters, mostly the vertical load and the radius of curvature. Aydın (2025) proved that the radius of curvature, vertical load, and friction characteristics are crucial factors in the design of Friction pendulum systems by thoroughly analyzing the curvature calculations, manufacturing processes, and engineering applications of these systems [3]. The inability of conventional sliding isolation systems to sustain tensile demands can constitute an important design limitation, particularly when seismic overturning effects generate uplift forces at the isolation level. On the other hand, LRB systems offer horizontal stiffness and damping properties, as well as the ability to withstand vertical tensile pressures to a certain extent due to the composite action of rubber and steel layers.

In some isolation bearings, overturning moments especially those produced during earthquakes can result in both compressive and tensile strains. This behavior is particularly noticeable in systems with strong seismic demands and relatively moderate vertical loads, as well as in tall structures. Elastomeric bearings can sustain their integrity up to a specific stress level because of the rubber layers' capacity to work under tension. Tensile forces, however, are a crucial design consideration for choosing an isolation system because they can cause major changes in bearing behavior if the cavitation limit which is linked to rubber's tensile behavior is surpassed. The allowed tensile stress limit of 1.0 MPa (code acceptance criterion), as stated in ASCE 7 [1], was confirmed in this investigation, and the cavitation-related tensile capacity of LRB bearings was assessed to be in the range of roughly 1.5–2.5 MPa (physical cavitation threshold). As a result, LRB systems have a considerable advantage in system selection since they have a specific tensile capability, in contrast to FPS isolators do not provide tensile resistance in the conventional sense and may experience loss of contact under uplift demands.

According to study examining the behavior of LRB and 3D seismic isolation systems with respect to tensile forces done by Kılıç et al. (2022), the seismic behavior of liquid storage tanks with 2D and 3D base isolation systems was investigated. The effects of these systems on base shear force, tank wall stresses and deformations, sloshing behavior, and tensile forces produced within the isolation units were assessed [8]. In particular, the study showed that 3D elastomeric isolation devices function better in preventing tension-related problems. This result suggests that when choosing isolation devices, controlling horizontal movement is insufficient; vertical tensile demands that are anticipated to occur during an earthquake must also be considered. As a result, during the initial design stage of the current study, tensile capacity and the cavitation limit were regarded as crucial selection factors in the comparison of FPS and LRB systems.

The objective of this study is to perform the seismic isolation design of a six-story reinforced concrete structure in accordance with ASCE 7 [1]. Preliminary designs for LRB and FPS systems were created in the first phase of the project, and their basic seismic behavior traits were contrasted. The major criteria for choosing the final isolation system were the axial tensile forces that are likely to arise in the isolation units under earthquake load combinations and the tensile capacity that elastomeric bearings can give up to the cavitation limit; the LRB system was selected. Using the established LRB parameters, a three-dimensional ETABS [4] model of the structure was produced in the second stage. Seven pairs of earthquake data scaled in compliance with ASCE 7 [1] guidelines were used for Nonlinear Time History Analysis and Response Spectrum Analysis. The base shear coefficient transferred to the superstructure was found to decrease to about 9% at the DBE level. Additionally, it was found that all isolation units' axial tensile stresses were below both the level that posed a cavitation risk and the 1.0 MPa limit allowed by ASCE7 [1]. The findings show that the LRB system greatly lowers structural seismic demands, which helps the superstructure stay within elastic behavior limits. They also show that the risks of tension and cavitation in the units can be effectively controlled under suitable design conditions.

2. Case Study: Seismic-Isolated Design of a Six-Story Reinforced Concrete Building According to the ASCE 7 Standard

2.1. Structural Definition

In this research, the seismic isolation design of a six-story reinforced concrete building was carried out. All story heights were taken as 3.0 m. The dead and live loads were assumed to be 25 and 10 kN/m², respectively. The columns and beams had cross-sectional dimensions of 30 × 60 cm and 30 × 50 cm, respectively. Slab thickness was selected as 15 cm. C25 concrete and B420C reinforcing steel were used. The structural mass and element dimensions were modeled using the ETABS software [4]. Maximum Considered Earthquake (MCE) Ground motion parameters were determined as Peak Ground Acceleration (PGA)= 0.35g, S_s = 1.5g and S_1 = 0.75g; Design Basis Earthquake (DBE) Ground motion parameters were determined as Peak Ground Acceleration (PGA)= 0.25g, S_s = 1.0g and S_1 = 0.50g for Site Class C conditions in accordance with the ASCE 7-22 standard [1]. The structure was checked for A1 torsional irregularity, and it was observed that the maximum irregularity factors which were equal to 1.045 and 1.041 remained below the limiting value of η_{bi} 1.2. As load combinations, 1.4D, 1.2D + 1.6L, 1.2D + L + E and 0.9D + E were used by considering ASCE 7 [1]. D, L and E represent dead load, live load and earthquake loads respectively. Fig. 1 presents the isometric view, plan view, and deformed modal shape of the ETABS [4] model.

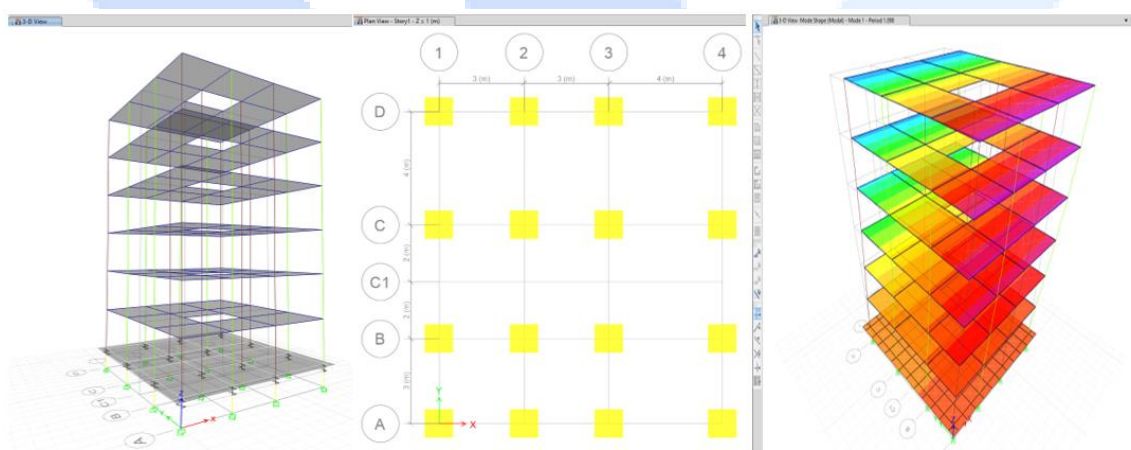


Figure 1. Isometric view, plan view, and deformed modal shape, of ETABS model [4]

2.2. Preliminary Design and Isolator Type Selection

During the preliminary design phase, Lead Rubber Bearings (LRB) and Friction Pendulum System (FPS) isolators were sized and compared as alternatives. According to ASCE 7-22 [1], several applicability criteria were considered in the preliminary design. The mapped spectral response acceleration parameter S_1 was 0.50 g, which is below the specified limit of 0.60 g. The site class was assumed to be Class C, and the building height was 18 m, which is below the 20 m limit considered for the isolation system. Furthermore, soil class can be A, B, C, D for isolated structures, here it was C. Additionally, maximum height of the structure should be below 20 m. This structure was 18 m. Axial tensile forces occur in the isolators beneath corner columns under seismic load combinations ($0.9D + E$). Even though, tension in isolator is allowed according to ASCE 7 [1], while FPS isolators cannot transmit tensile stresses, LRB isolators can withstand tensile forces up to the cavitation limit (1.5 – 2.5 MPa) due to the nature of the elastomeric material. ASCE 7 standards [1] permit tensile stress of up to 1.0 MPa in isolators. Calculations determined the maximum allowable tensile force for an isolator with a radius of $R = 400$ mm to be 502.65 kN, leading to the final selection of the rubber-type isolator. While the first-mode period of the non-isolated structure was $T_1 = 1.098$ s, the addition of the isolation layer shifted the effective period to $T_{eff} = 2.327$ s. The Design Earthquake (DBE) and Maximum Considered Earthquake (MCE) spectrum were scaled using a damping coefficient of $BD = BM = 1.5$, accounting for a 20% damping ratio with respect to ASCE 7 [1] Table 17.5.1 as given in Fig. 2. Fig. 3 compares the response spectrum for 5% and 20% damping ratios and presents the corresponding displacement spectrum for the 20% damping case.

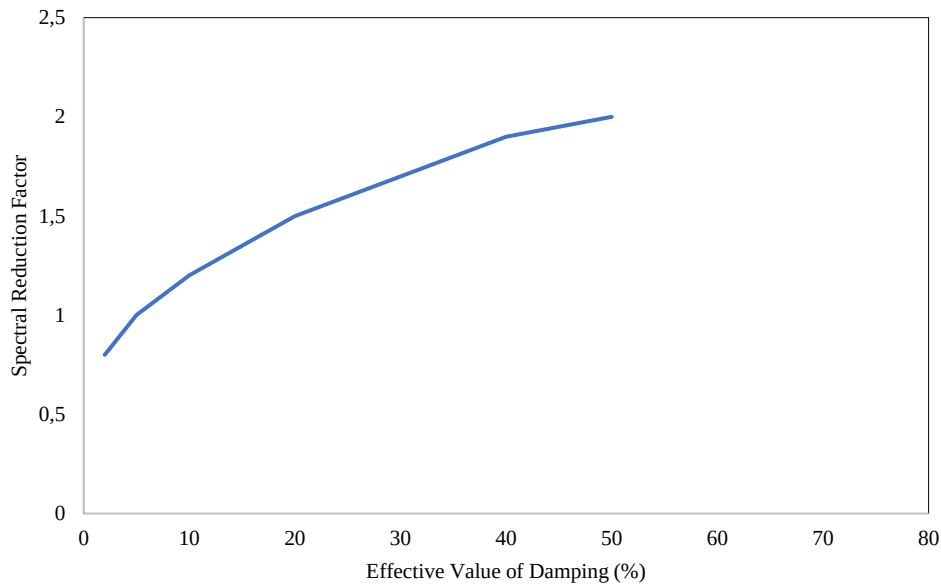


Figure 2. Damping coefficients (ASCE 7 [1])

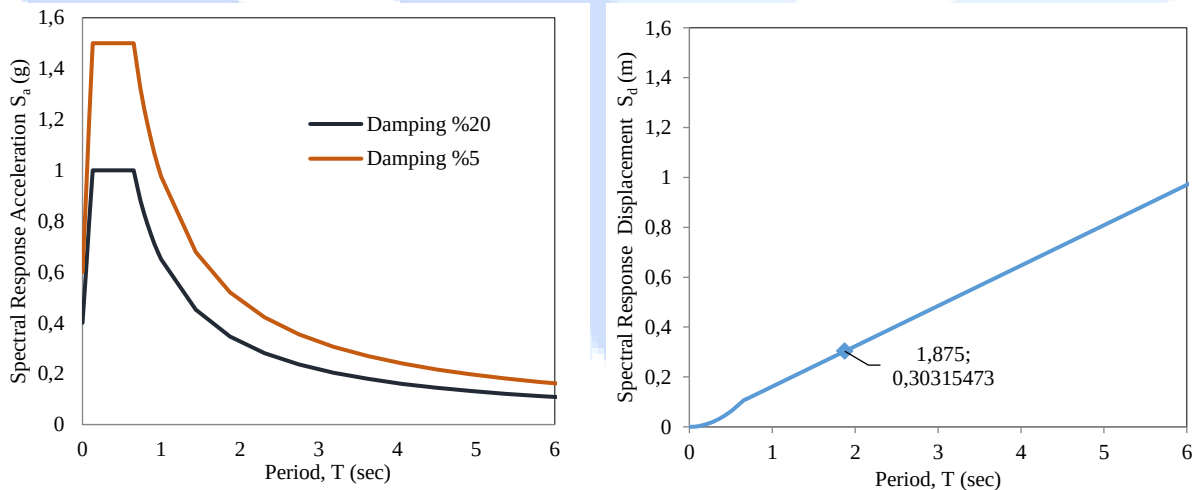


Figure 3. MCE response spectra (20% and 5% damping) and displacement spectra (20% damping)

The effective period and stiffness of the isolation system were subsequently determined based on the preliminary design calculations described below. The geometric and mechanical properties of the isolator were calculated for both isolator types as follows: Initially, Friction Pendulum System (FPS) calculations were completed. The effective radius of curvature of the isolators R_{eff} is 90 cm. The effective damping ratio is 20%, and the effective isolation period (TM) is 1.875 s. The maximum allowable horizontal displacement of the isolator DM for Maximum Considered Earthquake (MCE) is 30 cm. It should be noted that the calculated value of 1.875 s does not represent a fixed geometric pendulum period. Instead, it corresponds to the effective isolation period obtained for $\mu_{min}=0.01$ and $DM=300$ mm [15]. The minimum effective stiffness $k_{M, min}$ is calculated

by considering the vertical force (W) and the dynamic friction coefficient μ_{min} (minimum value at zero displacement) acting on the isolator, as stated in Eq.1.

$$k_{M,min} = \frac{\mu_{min}W}{D_M} + \frac{W}{R_{eff}} = \frac{0.01W}{0.3} + \frac{W}{0.9} = 1.144W \quad (1)$$

The effective period T_M for the maximum displacement shall be calculated with Eq. 2 [1].

$$T_M = 2\pi \sqrt{\frac{W}{k_{M,min}g}} = 2\pi \sqrt{\frac{W}{1.144W \cdot g}} = 1.875 \text{ s} \quad (2)$$

The displacement of isolation system, D_M , can be calculated with DM Eq. 3. Here g , S_M , T_M and B_M refer to acceleration of gravity, MCE 5% damped spectral acceleration, effective period and numerical coefficient related to the effective damping of the isolation system at the maximum displacement [1].

$$D_M = \frac{g S_M T_M}{4\pi^2 B_M} = \frac{9.81 \cdot 0.52005 \cdot 1.875}{4\pi^2 \cdot 1.5} = 0.302 \text{ m} \approx 0.3 \text{ m} \quad (3)$$

The effective damping ratio β_M will be calculated with Eq.4. The corresponding damping coefficient B_M is obtained from ASCE 7-22 [1] Table 17.5-1.

$$\beta_M = \frac{2}{\pi} \left(\frac{\mu_{min}}{\mu_{min} + \frac{D_M}{R_{eff}}} \right) = \frac{2}{\pi} \left(\frac{0.01}{0.01 + \frac{0.3}{0.9}} \right) = 0.185 \approx 20\% \rightarrow B_M = 1.5 \quad (4)$$

For Lead Rubber Bearing (LRB), the effective damping ratio is identical, and it is 20%, and the effective isolation period (T_M) is equal, and it is 1.875 s. Since the maximum allowable horizontal displacement of the isolator D_M for Maximum Considered Earthquake (MCE) is 30 cm as calculated in Eq.3, the effective damping ratio β_M can be determined with Eq. 4 again [1]. Additional calculations for Lead Rubber Bearing (LRB) were needed. Initially, required area and then diameter (D) of the isolator unit was determined with Eq. 5. P_{D+L} which refers to the maximum axial compression force on an isolator unit is equal to 395577 kgf. σ_c is the allowable compression stress and it is 80 kgf/cm².

$$\sigma_c = \frac{P_{D+L}}{A_0} < 80 \frac{\text{kgf}}{\text{cm}^2} \rightarrow A_0 = \frac{395577}{80} = 4944.7125 \text{ cm}^2 \rightarrow D = 79.346 \text{ cm} \approx 80 \text{ cm} \quad (5)$$

The required diameter of the lead core was then determined. The total characteristic strength of the isolators was selected to resist 10% of the total seismic weight which is equal to 28989.3 kN. There are 16 columns and isolator unit. Q is the total yield force of isolator and can be found from Eq. 6. Here, τ_{yp} is the yield stress of the lead plug according to the study done by Mayes and Naeim (2001) [12]. A_p is the required area, and D represents the diameter of lead part.

$$Q = 0.1 \cdot W = 0.1 \cdot 28989.3 = A_p \times \tau_{yp} \rightarrow \frac{2898.93}{16} = 181.183 \text{ kN} = 10 \times A_p \rightarrow A_p = 181.18 \text{ cm}^2 \rightarrow D = 15.19 \approx 16 \text{ cm} \quad (6)$$

Following these calculations, the nominal force-displacement response of a single LRB, together with its upper and lower bounds, was determined as shown in Fig. 4. While calculating upper and lower bounds, $\Lambda_{bottom}=0.765$, $\Lambda_{top}=1.84$ coefficients were used. Single unit has 15 pieces of 10mm thickness rubber layers and 14 pieces of 3mm steel plates. Total height of LRB units becomes 192mm. In Fig. 5, isolator section views and definition in ETABS [4] are given.

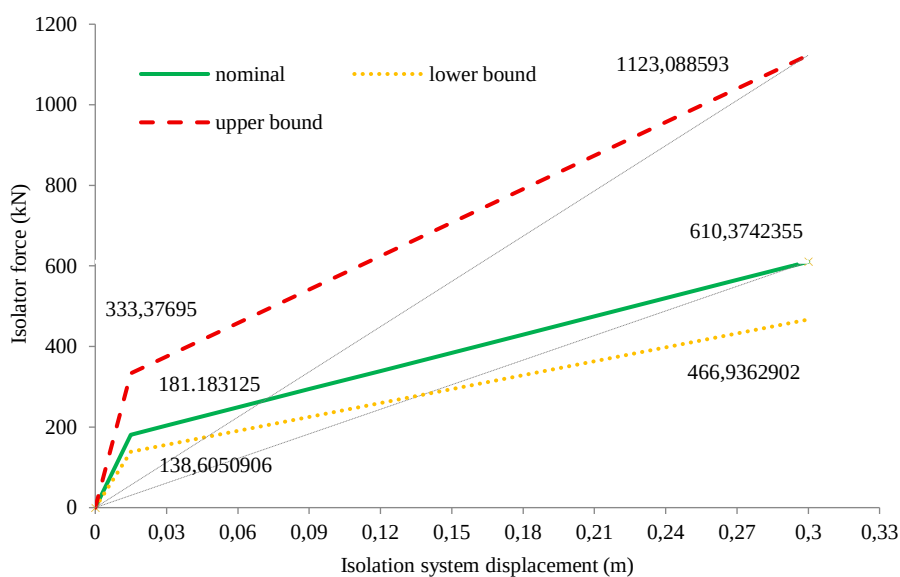


Figure 4. Single LRB nominal behavior with upper and lower bounds (ASCE 7) [1]

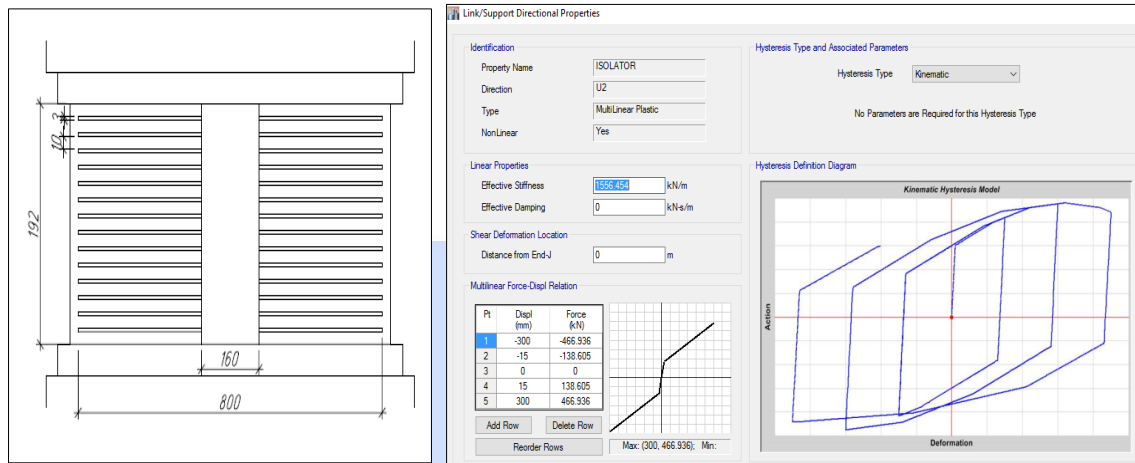


Figure 5. Isolator section view and definition in ETABS [2, 4]

3. Dynamic Analyses and Results

Using the final isolator properties, nonlinear time-history and response spectrum analyses were performed using seven bidirectional ground-motion record pairs were selected, resulting in 14 horizontal components selected from the PEER database and scaled in accordance with ASCE 7-22 [1,13]. In Fig. 6., MCE ground motion records' spectra in x and y direction are shown. DBE and MCE response spectra are given in Fig.7. The peak ground acceleration is represented by PGA, while Mw denotes the magnitude of the earthquake.

Table 1. Key parameters of earthquake ground motions from PEER database

Name of Record	Mechanism	Year	Station	Magnitude (M_w)	PGA (g)
2699CHICHIO4.CHY024	Thrust	1999	TCU068	7.7	0.361
2893CHICHIO4.TCU122	Thrust	1999	TCU075	7.7	0.361
1614DUZCE.1061	Strike Slip	1999	Station 8101	7.2	0.520
1618DUZCE.531	Strike Slip	1999	Station 1401	7.2	0.520
164IMPVALL.H-CPE	Strike Slip	1979	El Centro Array	6.9	0.315
838LANDERS.BRS	Strike Slip	1992	Barstow	7.3	0.780
879LANDERS.LCN	Strike Slip	1992	Lucerne Valley	7.3	0.780

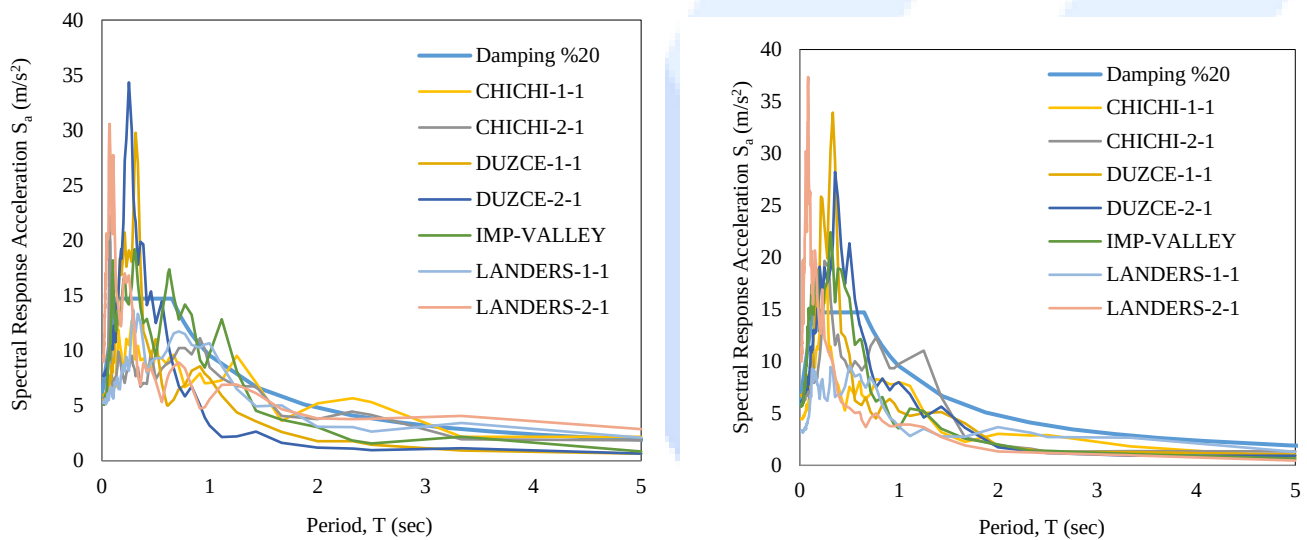


Figure 6. MCE ground motion records' spectra in x and y direction

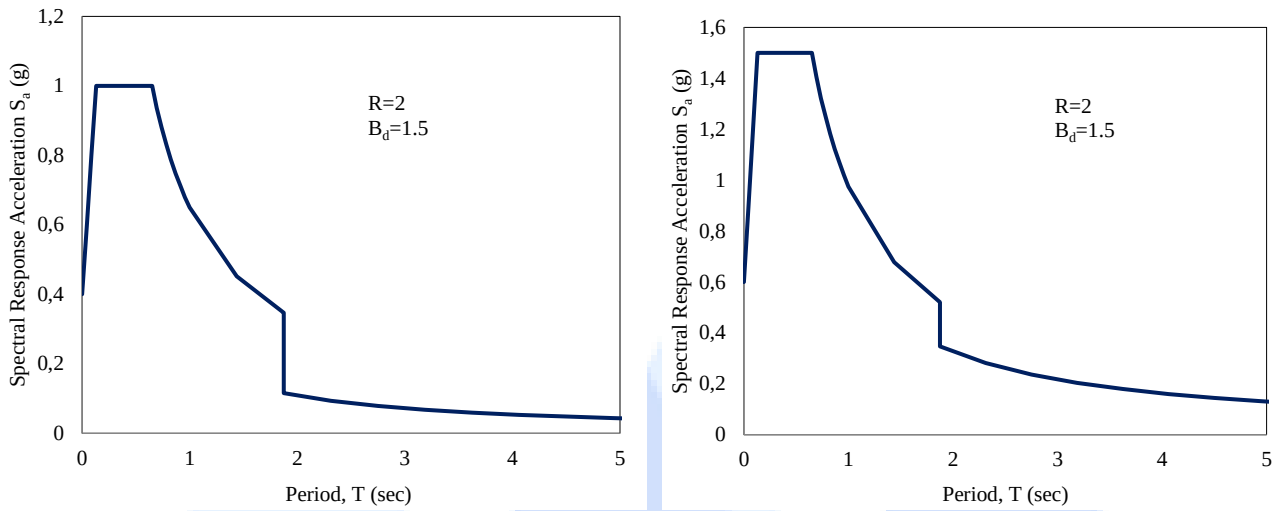


Figure 7. DBE and MCE response spectra

3.1. Base Shear, Hysteretic Response Curves and Isolator Displacements

Design Basis Earthquake (DBE) response spectrum was used for the determination of story shear of floors above the isolation system. The upper-bound isolator properties were used for the evaluation of the superstructure story shear forces. Maximum Considered Earthquake (MCE) response spectrum was used for comparison of drift and displacement values for the NTHA results with lower bound values of isolators. Response spectra plot obtained from Düzce 2-1 record for y direction and an example joint-1 displacement for two directions is illustrated in Fig.8. According to the base isolated structure's modal analysis, the first-two-mode periods, which were 1.098 and 1.081 seconds in the first state, increased to 2.327 and 2.321 seconds when the isolation system was added.

Table 2. Max-Mean time history displacements (SRSS) checks

Name of Record	Fault Normal (mm)	Fault Perpendicular (mm)
2699CHICHIO4.CHY024	405.521	397.702
2893CHICHIO4.TCU122	333.749	330.629
1614DUZCE.1061	149.010	146.784
1618DUZCE.531	146.849	148.262
164IMPVALL.H-CPE	194.354	195.139
838LANDERS.BRS	251.961	257.128
879LANDERS.LCN	452.606	457.165
MAX-MEAN	276.293	276.116

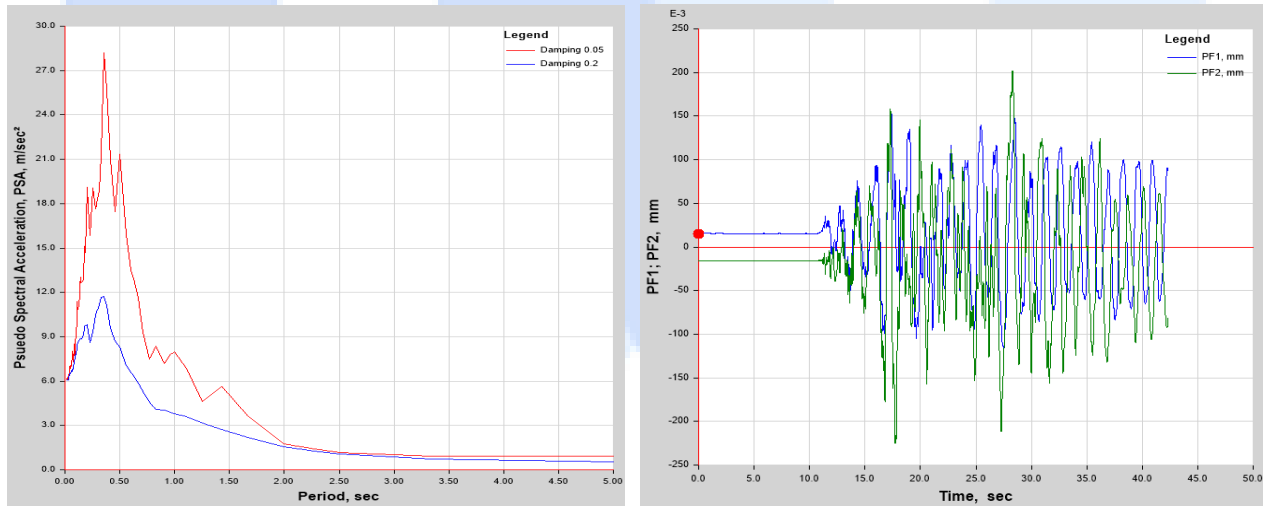


Figure 8. Response spectra plot obtained from Düzce 2-1 record for y direction and an example joint-1 displacement for two directions [4]

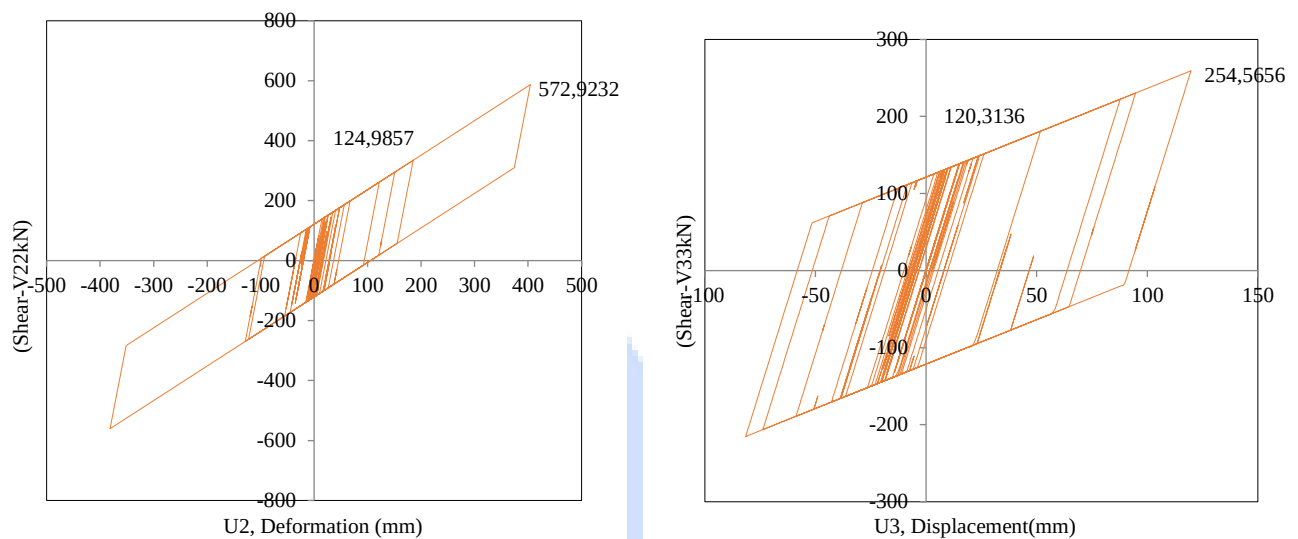


Figure 9. Link object hysteresis response output for Imperial Valley load case for both direction (63.74 sec)

As an example, link object hysteresis response output for Imperial Valley load case for both directions are shown in Fig.9. In Table 2. Max-Mean NTHA displacements (SRSS) checks are indicated above. In the response spectrum analysis, the average peak displacement was found to be 295.9 mm; it was verified that NTHA results exceeded 90% of the spectral results (276.2 mm). The maximum mean displacement obtained from the nonlinear time-history analyses was 276.2 mm, corresponding to 93.3% of the 295.9 mm displacement obtained from the response spectrum analysis. Under the Design Basis Earthquake (DBE), the base shear force acting on the stories above the isolation layer was found to be V_{base} 2004 kN, representing approximately 9% of the total structural weight ($W = 28989$ kN). The hysteretic response of a representative isolator was compared with the predefined isolator force-displacement relationship in the ETABS model [4]. The numerical results show close agreement between the two responses. The square-root-of-the-sum-of-the-squares (SRSS) method was used to combine the orthogonal displacement components.

3.2. Inter-Story Drifts

Fig. 10 presents the interstory drift ratios in both principal directions obtained from the MCE response spectrum analysis. Figs. 11–14 present the corresponding maximum and minimum interstory drift ratios obtained from the nonlinear time-history analyses in the two principal directions. In the graphs, level 0 refers to foundation. Levels 1 and 2 refer to bottom and top of isolator unit. Remaining six levels are for the stories above the base isolation. Maximum inter-story drift ratios in the nonlinear history analyses were slightly higher than those in the response spectrum analyses. The maximum interstory drift ratio was 1.6%, which remained below the applicable ASCE 7-22 limit of 2% [1].

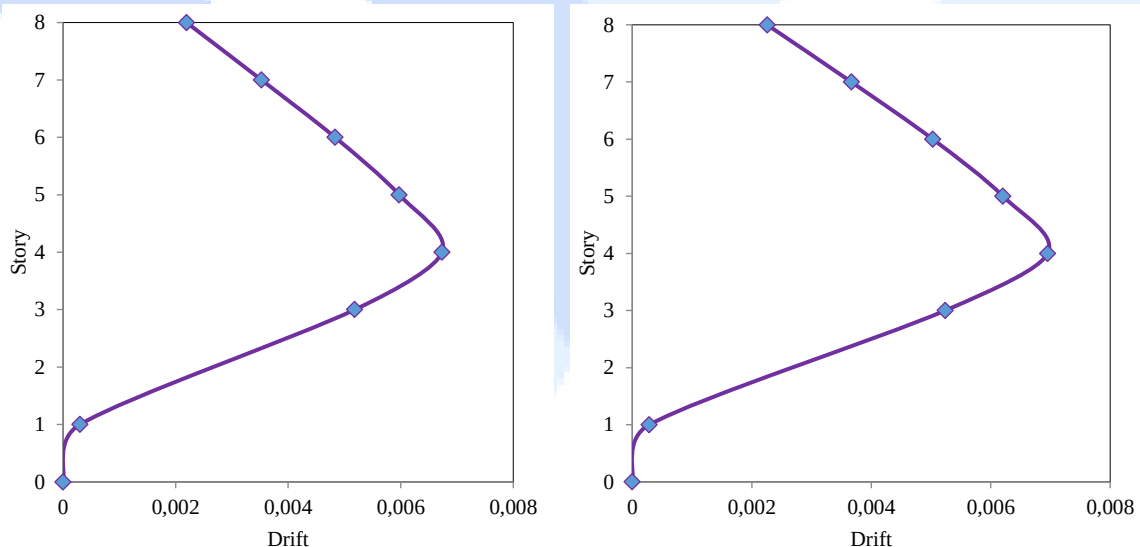


Figure 10. MCE-Drift-X-Y according to response spectrum analysis

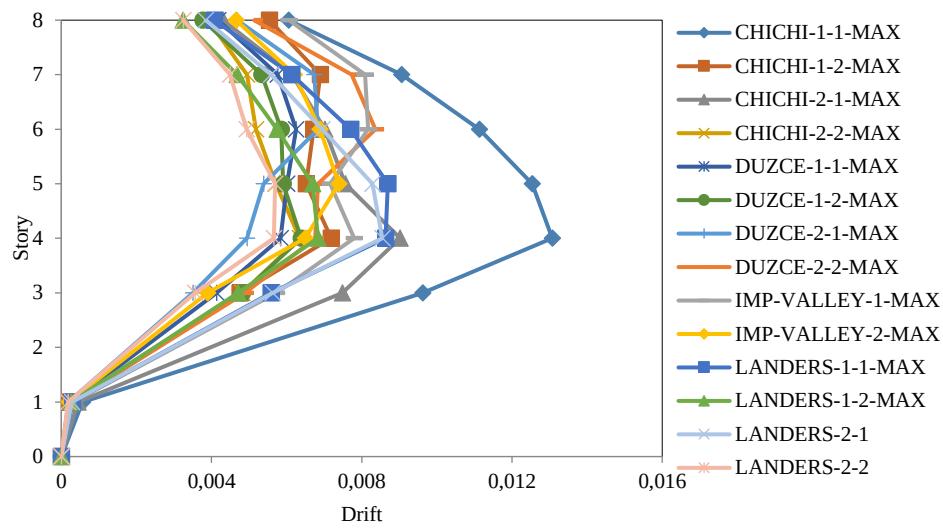


Figure 11. Drifts-Max-X

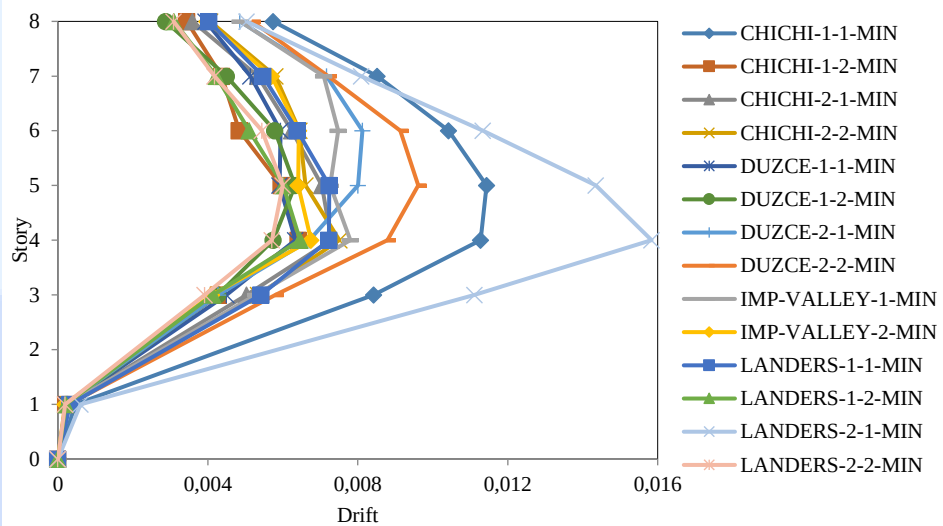


Figure 12. Drifts-Min-X

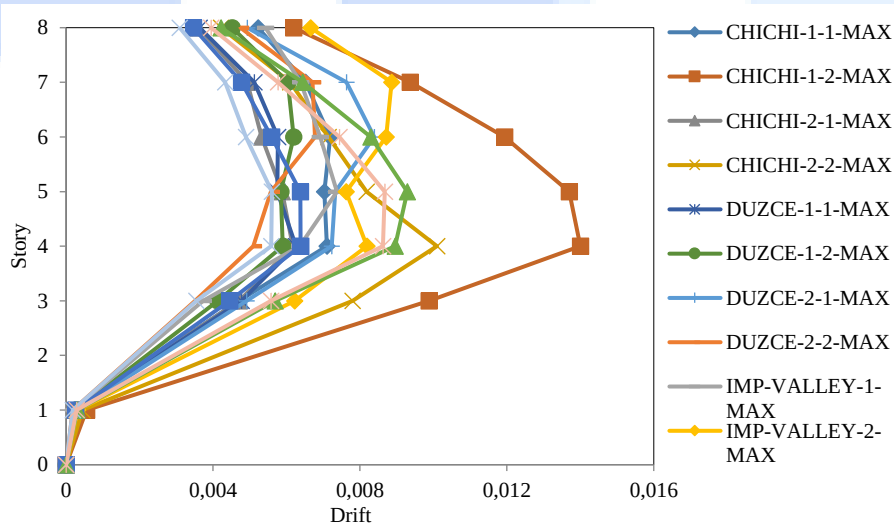


Figure 13. Drifts Max-Y

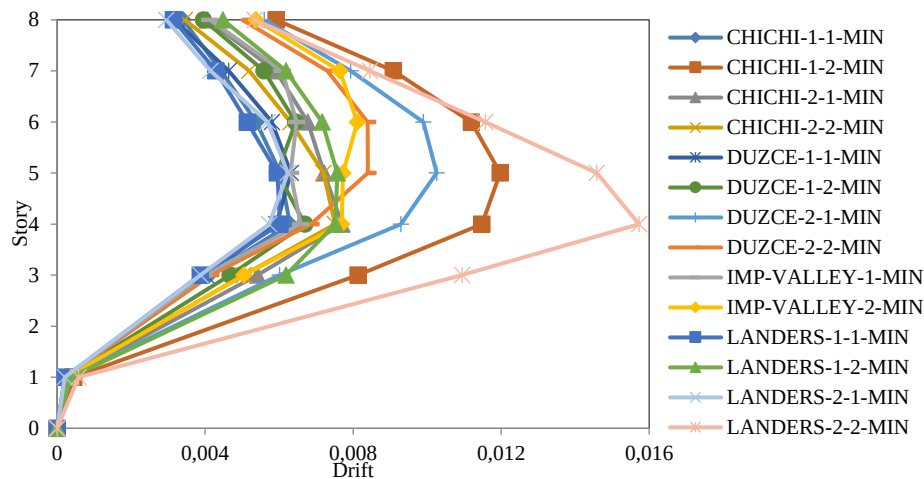


Figure 14. Drifts Min-Y

3.3. Isolator Tension Checks

Cavitation may occur when the tensile stress in an elastomeric bearing reaches a critical level, reported in the range of approximately 1.5–2.5 MPa. In contrast, EN 15129 and TBEC 2018 [6,14] do not permit tensile demands in elastomeric isolation bearings, whereas ASCE 7-22 [1] permits a tensile stress of up to 1.0 MPa under the specified design conditions. Maximum allowable tension depends on the cross-sectional area. Monitoring of the axial forces in the critical isolators showed that the tension limit of 1.0 MPa (502.65 kN) calculated with Eq. 7 was not exceeded under any earthquake record, and no risk of cavitation occurred [9,11]. If FPS type isolators were used instead, there would be uplift.

$$\text{Tension Capacity} = 1\text{MPa} * (\pi r^2) = 1 * (\pi * 400^2) = 502655\text{ N} = 502.655\text{ kN} \quad (7)$$

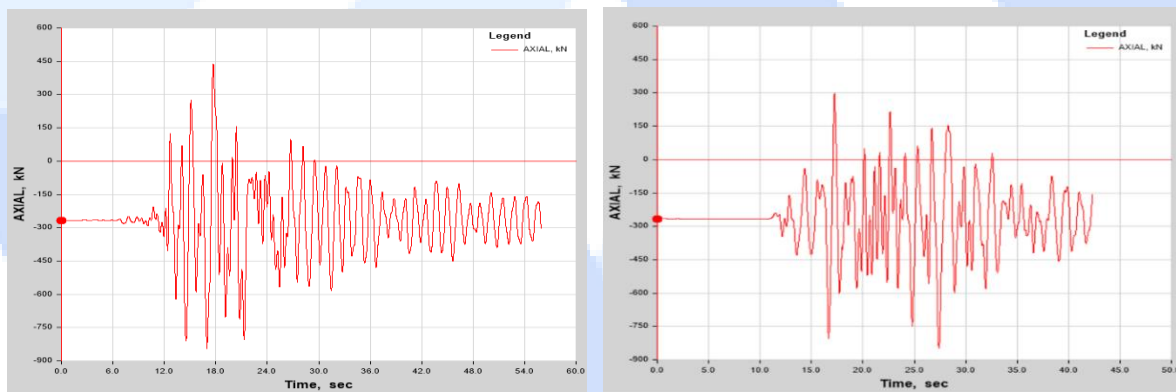


Figure 15. Axial link force time history response (CHICHI-2-1 case for maximum tension DUZCE-1-2 case for maximum compression) [4]

4. Conclusion

In this study, nonlinear time-history and response spectrum analyses were used to examine the seismic base-isolated behavior of a six-story reinforced concrete structure built in compliance with the ASCE 7 standard. Lead Rubber Bearing and Friction Pendulum Systems were first compared using preliminary design criteria; the LRB system was ultimately chosen after axial tensile forces from overturning moments, and cavitation restrictions were considered. The following is a summary of the main conclusions drawn from dynamic analyses utilizing seven scaled earthquake records:

- After the isolation layer was added, the fixed-base structure's (without base isolation) first-mode natural vibration period which was 1.098 seconds rose to 2.327 seconds. The increase in the fundamental period shifted the structural response toward a lower spectral acceleration range.
- The base shear coefficient acting on the superstructure above the isolation layer decreased to about 9% (2004 kN) at the Design Basis Earthquake (DBE) level. This demonstrates that the seismic demands conveyed to the superstructure are greatly reduced by seismic base isolation.
- The Max-Mean displacement (SRSS) derived from time-history analysis at the Maximum Considered Earthquake (MCE) level was determined to be 276.2 mm. This result showed that at least 90% of the value derived from response spectrum analyses and stayed within the 300 mm limit estimated during preliminary design.
- The maximum axial tensile stresses in the corner isolators under seismic load combinations (0.9D + E) stayed below the elastomeric material's cavitation risk threshold (1.5–2.5 MPa) and the 1.0 MPa (502.65 kN) limit allowed by the ASCE 7 standard.

- LRB isolators provide a measurable tensile resistance up to the cavitation limit, which constitutes an important advantage when uplift demands develop at the isolation level.

Considering the results obtained, the following recommendations are given for future studies:

- To comprehend how vertical earthquake components affect isolator axial load changes and cavitation risk, future research on 3D seismic isolation systems should be both mathematically and experimentally studied.
- A different parametric investigation should examine the effects of near-fault earthquake recordings, particularly the excessive displacement demands placed on the base isolation layer by near-fault records with strong pulse features and the related secondary P-Delta effects.
- Research should focus on how smart friction surfaces that are resistant to temperature and load variations or hybrid configurations which combine LRB and FPS isolators inside the same structure affect the structure's seismic performance.

The conclusions are limited to the investigated six-story reinforced concrete building, the adopted site conditions, the selected ground motions, and the assumed isolator properties. Therefore, the observed superiority of the LRB system in terms of tensile resistance should not be generalized to all isolated structures without considering structural configuration, axial load distribution, overturning demands, and isolator-specific properties.

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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An Analysis of High Urbanization via Clustering in South Korea for Regional Integration

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Keywords

Clustering calculation,
Urban,
High-level transport,
Tourism,
Feasibility.

Abstract

High-speed transportation plays a significant role in enhancing regional accessibility, supporting economic development, and strengthening social and cultural connections between urban centers. In this context, identifying metropolitan areas and evaluating their relative development levels are essential to establishing effective, sustainable transportation networks. This study investigates the potential of clustering analysis to identify priority urban centers in South Korea and to develop a framework for regional integration through high-speed transportation. We used population, gross domestic product (GDP), and city livability indicators to evaluate the relative characteristics of selected South Korean cities. K-means, Partitioning Around Medoids (PAM), and Clustering Large Applications (CLARA) methods were applied using the R-Studio statistical computing environment. The clustering results were examined to identify dominant metropolitan centers and groups of cities with similar socioeconomic characteristics. Based on these findings, a high-speed transportation network was proposed to strengthen connections between major urban areas and their surrounding regions. The study also discusses the potential contribution of high-speed mobility to tourism development, regional accessibility, and long-term metropolitan integration. The proposed clustering-based framework provides a preliminary decision-support approach for prioritizing urban centers and developing a more integrated transportation system in South Korea. Further feasibility studies incorporating travel demand, infrastructure capacity, operating costs, and environmental considerations are recommended to support the implementation of the proposed network.

1. Introduction

In this context, transport infrastructure should not be evaluated only as a physical connection between settlements. It also functions as a mechanism for concentrating labor, knowledge, investment, tourism demand, and public services within a connected urban system. Accordingly, the identification of dominant cities through population, economic output, and livability indicators can support a more systematic interpretation of regional integration. A cluster-based approach is particularly useful because it groups cities with comparable characteristics and helps distinguish primary metropolitan centers from secondary and peripheral urban areas. This distinction is important for determining where high-capacity transport links may generate the greatest regional benefit.

Transportation is at the heart of development, attracting tourism, social activity, economic growth, and cultural relations. Transportation services increase the emphasis on tourism travel at city, regional, national, and international levels. Today, tourism is one of the main pillars of economic development for countries.

Multi-sectoral improvement is a challenging issue, especially for developing countries. Furthermore, these developing nations strengthen core sectors such as construction, healthcare, tourism, heavy industry, infrastructure, energy, military, or light industry. The driving force behind tourism is a well-developed transportation system. High-speed systems and well-planned urban clustering's are important asset, capacity, and opportunity for national based tourism.

Korea is among the countries with the oldest history in the world. Korea in East Asia has ethnolinguistic ties with peoples such as the Mongols, Manchus, Turkic peoples of Asia, and the Japanese. South Korea has an area of 100.363 km² and a population of 51,641,642 as of the end of 2025 (Worldometer, 2025).

A large portion of the Korean population is ethnically Korean. The country was divided into South and North in 1948, immediately after World War II. Between 1950 and 1953, there was a major war and destruction between North Korea and South Korea, reaching international dimensions and motivated by the Cold War. There is also a Turkish population in South Korea dating back to those war years. North Korea's population is

26.606.699 and its land area is 120.540 km² (Worldometer, 2025). The integration of the two countries is an important issue, but there is currently no such political possibility.

In South Korea, over half the population is atheist, while in North Korea, this figure reaches 73%. In South Korea, 15% of the population is Buddhist, while in North Korea, 12% are Shamanist. Additionally, 27% of South Koreans are Christian (İnsamer Ülke Profili Güney Kore, 2025). The Korean Peninsula is bordered by China's Manchuria region to the north. Its sea neighbor is Japan. Korea borders the Pacific Ocean. South Korea has a developed infrastructure and is among the developed countries. The average lifespan in South Korea is 82.5 years. South Korea's transportation system and public transport are of a high standard. The country also has a high-speed train network. However, the country's air and sea integration with Japan is quite important. Relations with China's Manchuria region are also an economically important and decisive factor for the country.

Development of the region will start with integration, which is strongly bounded to infrastructural and metropolitan integration and advancement. Within the scope of this study, cities of South Korea were listed in terms of Dynamics like population, livability indexes and gross domestic products (GDP's), and were subjected to clustering analysis using statistical packages within the R Studio environment. On this parallel, priority and important urbans were specified by determining the number of clusters and cluster lists that expressed rely on the most adaptable cluster method. These priority cities were re-evaluated and analyzed based on operational distances, and lastly accessed to a high-speed mobility system proposal and metropolization roadmap for South Korea.



Figure 1. Big Urbans of South Korea Map (World Map, 2025)

North Korea, Manchuria Region (China), Amur (Russia), Kamchatka (Russia), Japan, Vietnam, Philippines and Thailand are significant parts of this mentioned region. The total of all of the called nations substitutes a quite huge land area with their various important sources, population, population density, cultural variety, urban developing and efficient geographical conditions too. And addition to all of these, there are some significant and large islands around this mentioned and specified region on Pacific Ocean too.

This region also has a neighborhood with far Asia, especially with China, east Russia, Asia Pacific islands and Vietnam as a whole manner. So, the cultural, geographical and social features of this area are quite deep and enough rich. This composition and photo could be meaningful with detailed mapping. This mentioned geography has a high priority and urgency for integration and development because of strategical conceptual.

In Figure 2 South Korea state map is presented (BBC Asia Pacific, 2025). Figure 3 shows South Korea Geographical Map (Worldometers Info, 2025). Table 1 shows South Korean cities' populations, live abilities and GDP's (OECD, 2020).



Figure 2. South Korea State Map (BBC Asia Pacific, 2025)



Figure 3. South Korean Geographical Map (Worldometers Info, 2025)

Table 1. South Korean cities populations, GDP's and liveabilities (OECD, 2020)

City	GDP	Population	Livability Index
Seoul	1068311	23736409	94
Gimhae	150935	4454748	73
Dalseong	64228	2387450	72
Gwangsan	51429	1547046	72
Seo	48872	1449628	75
Seongsan	42108	1092468	66
Ulsan	61736	902534	72
Heungdeok	46220	879406	69
Deokjin	25928	745770	68
Sebuk	41273	655292	67
Gyeongsan	13179	264627	68
Pohan	22276	501058	71
Gumi	26008	526330	66
Jeju	12034	489241	64
Jinju	13291	346380	69
Wonju	12772	354049	65
Iksan	9664	278554	67
Chuncheon	10932	281862	70
Gunsan	8309	264798	67
Suncheon	14108	280802	67

2. Literature Review

In this century, besides high and rapid urbanization, digitalization and sustainability will also be at the forefront. The increasing trend of digitalization is a continuous, robust, and reliable process of technical improvement and innovative applications. This has emerged as a basic and robust dynamic of change in today's life, daily habits, and consumer culture. Mobility and metropolization have been and will continue to be shaped, influenced, and featured by these dynamics.

Sustainability trends are expected to become stronger in the near future. Western countries and developed and developing nations in Far Asia have a significant and effective role to play in this regard. In parallel, sustainability, digitalization, and urbanization indicate that greater investment is required in high-speed train lines, Maglev train lines, and high-capacity, high-speed interregional rail lines. These three systems may provide an initial basis for autonomous mobility and advanced transportation planning in developing and developed nations.

The development of well-planned metropolitan areas and high-standard transportation systems is an important consideration in regional development. At the center of this process are effective planning, detailed analysis, and appropriate approaches to transportation and urban development (Givoni, 2006) [8]. High-speed transportation systems can contribute to improved accessibility, economic competitiveness, and stronger connections between metropolitan centers and surrounding regions. Therefore, the planning of high-speed transport networks should be considered together with urbanization trends, population distribution, economic performance, and quality-of-life indicators.

Previous studies have emphasized that high-speed rail is closely related to changes in spatial organization, accessibility, and the distribution of economic activity. Givoni (2006) [8] reviewed the development and impacts of modern high-speed trains and highlighted their role in transforming intercity accessibility and regional development. Kiziltas (2020) [9] discussed the relationship between high-speed railway systems, production, and economic development, supporting the consideration of transport accessibility as an important planning variable. In addition, Sanchez-Borras et al. (2010) [10] examined the competitiveness of high-speed trains in relation to rail access charges and alternative transport modes. These studies indicate that transport planning should consider not only travel speed but also economic competitiveness, connectivity, and the potential for integration between urban centers.

Recent literature has further examined the relationship between high-speed rail and regional development. Lee (2023) [11] addressed high-speed rail and regional uneven development through a literature review. This study is relevant to the present research because it highlights the importance of examining regional development differences when evaluating transportation infrastructure. The potential contribution of high-speed rail to regional integration should therefore be considered alongside the existing economic and demographic characteristics of cities. In this context, identifying urban centers with comparable development characteristics can provide a useful basis for evaluating the potential effects of transportation investments on regional development.

Yadav and Jin (2025) [12] investigated the impact of high-speed rail on urban regions in South Korea through a systematic literature review. Their work is particularly relevant to the geographical context of the present study, which focuses on South Korean cities and the development of an integrated high-speed transportation network. The review provides a basis for considering the relationship between high-speed rail and urban development in South Korea. Accordingly, the present study extends this perspective by using population, GDP, and livability indicators to identify urban clusters and discuss their potential roles in regional transportation integration.

Urban clustering provides a useful analytical framework for examining these relationships. By grouping cities according to their similarities in population, GDP, livability, and other socioeconomic indicators, clustering analysis can help identify dominant metropolitan centers and secondary urban areas. This approach allows researchers to examine the spatial distribution of development potential and establish a preliminary basis for prioritizing transport connections. K-means, PAM, and CLARA methods provide alternative approaches for identifying urban groups and comparing the consistency of clustering results.

In the context of South Korea, the integration of major metropolitan centers and regional cities is particularly important for the development of an efficient high-speed transportation system. The country has a developed transport infrastructure and a high level of urban concentration, but the distribution of economic activity and population among cities is not uniform. A clustering-based analysis can therefore contribute to the identification of cities that may function as strategic nodes in a more integrated national and regional transport network.

The present study combines urban indicators with clustering methods to identify priority cities in South Korea. Population, GDP, and city livability are used to evaluate the relative characteristics of selected urban centers. The results are then interpreted in relation to metropolitan development, regional accessibility, tourism potential, and the possible contribution of high-speed transportation to long-term regional integration. In this way, the study aims to provide a preliminary framework for identifying transport priorities and supporting future feasibility studies for high-speed mobility systems.

3. Methodology of Clustering Analysis

Clustering statistical analysis have been conducted via K means method, PAM and Clara method. The indicator dataset is based on the city-level information reported by OECD (2020) [7].

3.1. K Means Method

Here are results for number of clusters in terms of K means method in Figure 4. The indicator dataset is based on the city-level information reported by OECD (2020) [7].

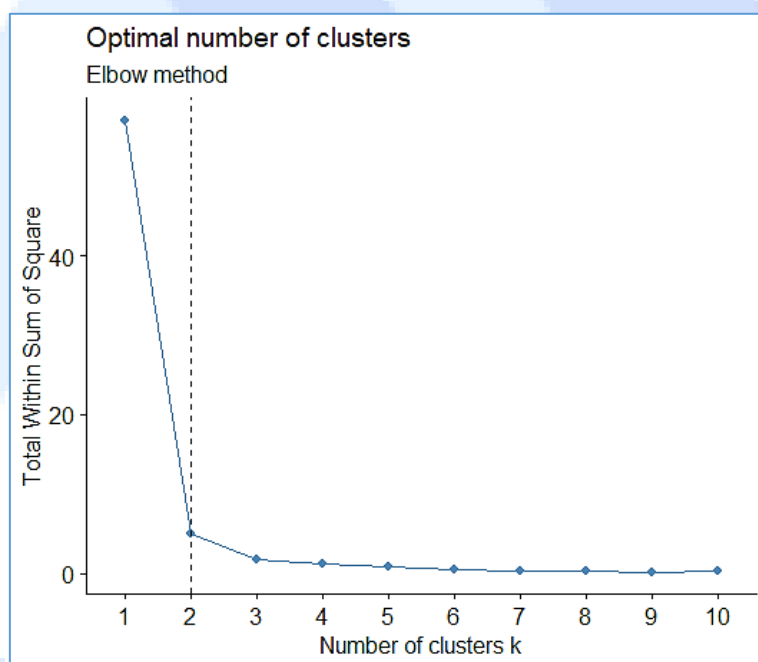


Figure 4. Number of clusters in terms of K means method, OECD (2020) [7]

Figure 5 illustrate Cluster Visualization in Terms of K Means Method.

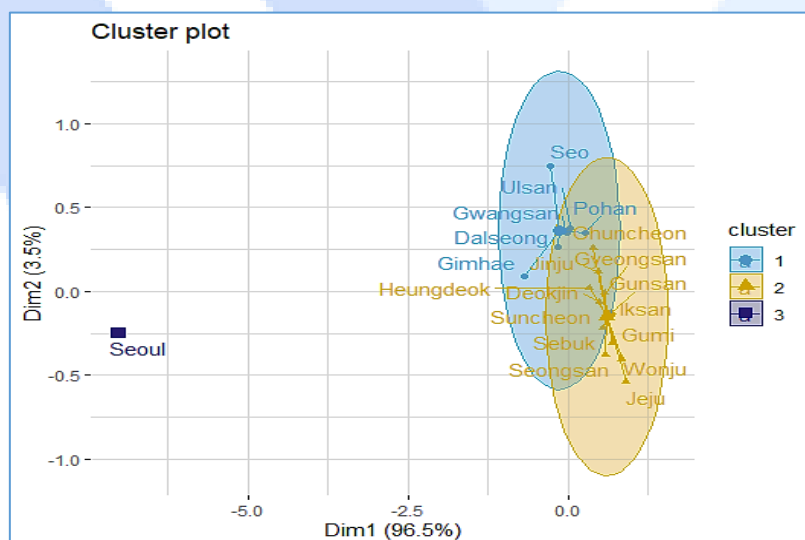


Figure 5. Cluster visualization in terms of K means method, OECD (2020) [7]

- **PAM Method**

Here are results for number of clusters and cluster Visualization in terms of PAM in Figure 6 and Figure 7.

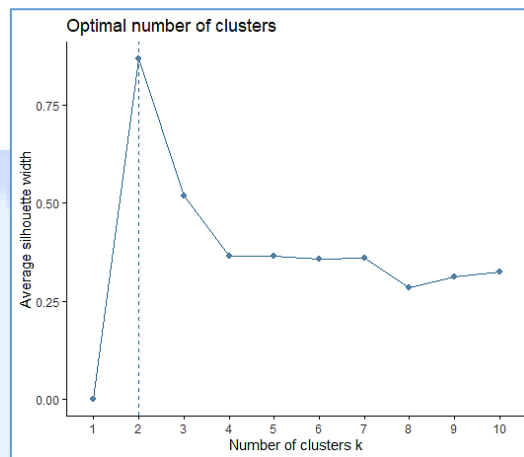


Figure 6. Number of clusters in terms of PAM

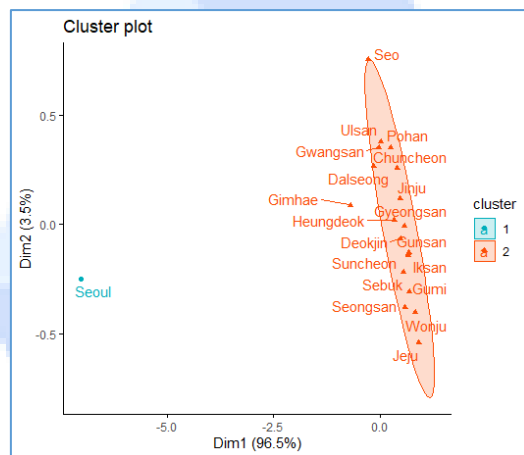


Figure 7. Cluster visualization in terms of PAM

- **Clara Method**

Here are results for number of clusters and cluster visualization in terms of Clara method in Figure 8 and Figure 9.

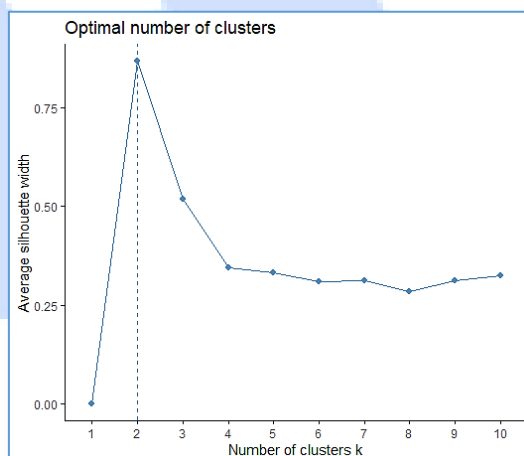


Figure 8. Number of clusters in terms of Clara method

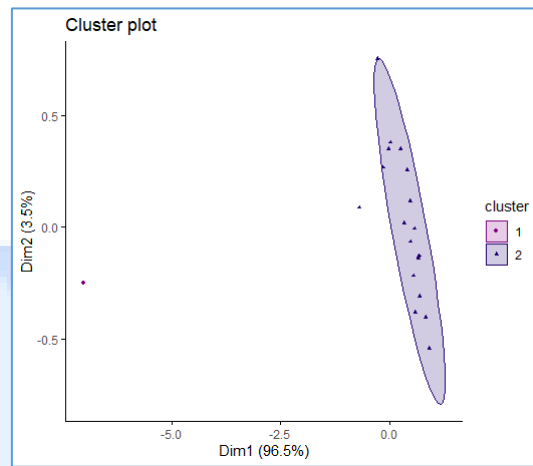


Figure 9. Cluster visualization in Terms of Clara Method

As a result of all methods and agglomerations, the biggest cities have been emerged into three clusters. On Table 2, clustering list of the cities has been given.

Table 2. Cluster list of south Korean cities

City	GDP	Population	Cluster
Seoul	1068311	23736409	3
Gimhae	150935	4454748	1
Dalseong	64228	2387450	1
Gwangsan	51429	1547046	1
Seo	48872	1449628	1
Seongsan	42108	1092468	2
Ulsan	61736	902534	1
Heungdeok	46220	879406	2
Deokjin	25928	745770	2
Sebuk	41273	655292	2
Gyeongsan	13179	264627	2
Pohan	22276	501058	1
Gumi	26008	526330	2
Jeju	12034	489241	2
Jinju	13291	346380	2
Wonju	12772	354049	2
Iksan	9664	278554	2
Chuncheon	10932	281862	2
Gunsan	8309	264798	2
Suncheon	14108	280802	2

4. Results and Discussion

After statistical analysis, prior urbans were gathered on 3-1 numbered clusters. On Table 3 selected cities are listed below.

Table 3. Selected cities via clustering analysis

City	GDP	Population	Cluster
Seoul	1068311	23736409	3
Gimhae	150935	4454748	1
Dalseong	64228	2387450	1
Gwangsan	51429	1547046	1
Seol	48872	1449628	1
Ulsan	61736	902534	1
Pohan	22276	501058	1

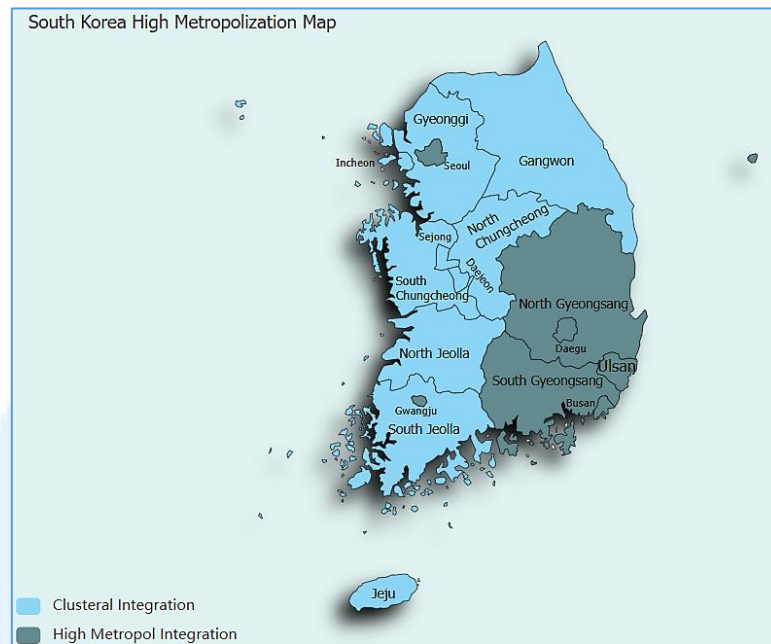


Figure 10. Metropolization Map of South Korea

The cities and urban areas selected above are regions that exhibit South Korea's highest level of metropolitanization and are trending towards producing the most advanced life mobility. Rely on the leading role these cities are going to play in region's economic, social, urban, and technological life in the short term, it is significant and a priority for region to be linked by a high-speed, high-capacity, and high-comfort inter urban connection. The development of this high-speed linkage will also serve as a new milestone of the national and trans nation regional transportation framework. On the other hand, constituting of a new high-capacity-high-comfort Transportation route based on determined cities and regions are going to play an important and reliable role on near and middle future integration improvements for South Korea, North Korea, Manchuria (China) and Amur (Russia) too.

Sustainability, urbanization, and digitalization are among the mobility phenomena of the 21st century. These phenomena can best take place in developed countries. Key indicators of these dynamics are high speed railways, Maglev trains, airway passenger transport and highway passenger transport. But for 400-600 km distances the best one is high speed railways. For 600-800 km distances, the best one is Maglev trains (Kiziltas, 2020).

Along with emerging technology, there are many alternative modes of transportation that provide high-quality service at high inter-city speeds. These can be classified as magnetically driven trains (Maglev), high-speed trains, airlines, high-comfort road transport. These modes of transport are highly competitive in cross-city travel, but their interconnections are combined or isolated accordingly.

- Competitiveness
- Integration
- Residence instead of each other
- Transformative transportation

Among the various cities in USA, this is the case with high-speed trains and airlines (Marta et al, 2010).

5. Conclusion

As shown in the Figure 10, the urbans proposed as a result of deep analysis in this paper are focused generally in coastal areas, South and Southeast parts of the country. These are main centers for integrating and unity of South Korea and peripheral. This area is not near to North Korea; it is on the vicinity of seaside and Japan. Seoul and Gwangju are city centers apart from south east coastal central area and they have unique properties. As it is known, Seoul is the capital city and will be the center of integration of north side with North Korea and South Korea on midterm future. Lastly Gwangju will be the center for future metropolization of south west side and integration of this part to Seoul and its peripheral.

Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Seismic Resilience and Hysteretic Damping of Nano-Silica Modified Basalt-Fiber Engineered Cementitious Composites: A State-of-the-Art Critical Review

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Keywords

Engineered Cementitious Composites (ECC),

Basalt Fiber,

Nano-Silica

Hysteretic Damping,

Energy Dissipation

Seismic Resilience

Abstract

Engineered cementitious composites (ECC), classified under strain-hardening cementitious composites (SHCC), exhibit remarkable pseudo-strain hardening and distributed tight microcracking behavior under mechanical stress, mitigating the brittle failure patterns typical of standard concrete. Basalt fibers (BF) provide an environmentally sustainable, high-tensile, and chemically stable mineral-based alternative to polymeric and metallic fibers. Concurrently, the inclusion of colloidal nano-silica (NS) refines matrix microstructures via intensified pozzolanic reactions and pore-filling mechanisms. This critical state-of-the-art review explores the synergistic role of nano-silica and basalt fibers in governing the seismic resilience of structural components subjected to reversed cyclic loading. Key dynamic performance indicators, such as hysteretic damping ratio, cumulative energy dissipation capacity, stiffness degradation profiles, and residual deformation metrics, are rigorously synthesized across multi-scale mechanical domains. Microstructural investigations focus on the interfacial transition zone (ITZ) refinement and crack-bridging mechanics controlling fiber rupture versus pullout failure modes. Methodological disparities, matrix brittleness risks arising from excessive nano-silica agglomeration, and conflicting experimental findings reported in current literature are systematically analyzed. Finally, unified evaluation frameworks and future research trajectories are outlined to facilitate the rational adoption of nano-modified BF-ECC in earthquake-resistant structural applications.

1. Introduction

Severe seismic excitations impose pronounced inelastic cyclic deformations on civil engineering infrastructure. Conventional reinforced concrete (RC) elements frequently suffer from brittle matrix degradation, extensive diagonal shear cracking, severe cover concrete spalling, and early rebar buckling under reversed cyclic hysteretic demands [1], [2], [3]. These damage patterns induce rapid stiffness degradation and severe pinching phenomena in hysteretic loops, markedly degrading structural hysteretic damping and energy dissipation efficiency [4]. Consequently, modern performance-based earthquake engineering has shifted its primary objective toward post-earthquake functionality, resilience, and strict damage tolerance [5].

To address the intrinsic brittleness of standard cementitious materials, Engineered Cementitious Composites (ECC)—or Strain-Hardening Cementitious Composites (SHCC)—were engineered based on micromechanical fracture mechanics [6]. Contrary to conventional fiber-reinforced concrete (FRC), which predominantly exhibits tension-softening behavior after first cracking, ECC demonstrates tensile strain-hardening capacities typically exceeding 2–5% with steady-state multiple microcracking [7], [8]. When incorporated into seismic-critical domains, such as plastic hinge zones of bridge piers, shear wall bases, and beam-column connections, ECC effectively mitigates damage localization, provides intrinsic confinement, and maintains high cyclic shear capacity [9], [10].

Historically, poly-vinyl-alcohol (PVA) and high-modulus polyethylene (PE) fibers have been the benchmark reinforcements for high-ductility ECC [11]. Nonetheless, ecological impacts, chemical durability constraints, and high material acquisition costs have stimulated research on inorganic mineral fibers [12]. Basalt fibers (BF), drawn from molten volcanic basalt rock, feature superior thermal stability, elevated tensile strength (up to 3000–4800 MPa), and competitive chemical resistance [13]. Recent breakthroughs have validated the feasibility of developing strain-hardening basalt-fiber ECC (BF-ECC) with saturated microcracking profiles and mean crack widths below 10–20 micrometers [14].

Concurrently, nanotechnology has redefined cement matrix optimization [15]. Incorporating nano-silica (NS) significantly alters hydration kinetics, densifying the interfacial transition zone (ITZ) between matrix and aggregates/fibers through accelerated pozzolanic reactions with calcium hydroxide (Ca(OH)₂) and secondary calcium silicate hydrate (C-S-H) gel formation [16], [17]. However, altering the matrix fracture

toughness and fiber-matrix interfacial frictional bond (τ_0) directly governs whether fibers undergo steady-state sliding pullout or premature rupture, determining composite ductility [18].

2. Review Methodology and Phenomenological Fundamentals

2.1. Analytical Framework and Selection Criteria

This critical review was executed using a systematic methodology targeting peer-reviewed publications across Scopus, Web of Science, and ScienceDirect between 2005 and 2026. The conceptual framework was focused on three distinct intersecting operational domains: (1) Basalt fiber micromechanics and bridging performance in cementitious matrices; (2) Microstructural matrix modification via colloidal and powdered nano-silica additions; (3) Cyclic hysteretic behavior, damping characteristics, and seismic performance of ECC structural subassemblies. Purely empirical concrete formulations lacking clear micro-mechanical characterization or cyclic behavioral validations were excluded to maintain analytical focus.

2.2. Micromechanical Criteria for Strain-Hardening Under Cyclic Actions

The transition from quasi-brittle fracture to pseudo-strain hardening in ECC is dictated by two fundamental micromechanical criteria established by Li and co-workers [6], [18]:

Strength Criterion: The composite first-crack stress (σ_{fc}) must not exceed the maximum complementary bridging stress (σ_0) across the crack plane:

$$\sigma_{fc} \leq \sigma_0 \quad (1)$$

Energy Criterion: The complementary bridging energy (J_b) must equal or exceed the matrix crack-tip fracture toughness (J_{tip}):

$$J_b = \sigma_0 \delta_0 - \int_0^{\delta_0} \sigma(\delta) d\delta \geq J_{tip} \approx \frac{K_m^2}{E_m} \quad (2)$$

Where $\sigma(\delta)$ represents the fiber bridging stress as a function of crack opening displacement (δ); δ_0 is the crack opening corresponding to maximum bridging stress; K_m is the matrix fracture toughness; and E_m is the composite elastic modulus. Under reversed cyclic loading, repetitive crack opening and closure induce frictional interfacial wear along the fiber embedded length, modifying chemical debonding energy (G_d) and frictional sliding stress (τ_0) [19].

3. Microstructural Modification Mechanisms of Nano-Silica

3.1. ITZ Refinement and Pore-Filling Mechanisms

The interfacial transition zone surrounding reinforcing fibers and micro-silica sand represents the primary shear-weak region in conventional cementitious matrices [16]. The introduction of reactive nano-silica particles (10–50 nm) counteracts this via three coupled mechanisms: (1) Physical Nucleation and Packing, which fills sub-micron interstitial spaces; (2) Intensified Pozzolanic Conversion, which consumes brittle $\text{Ca}(\text{OH})_2$ to produce rigid C-S-H gels; and (3) Fiber-Matrix ITZ Densification, which raises the slip friction coefficient during mechanical pullout [17], [21].

3.2. Critical threshold and agglomeration penalties

While moderate dosages of nano-silica (1.0–2.0 wt.% of binder) optimize stress-transfer, excessive concentrations trigger severe agglomeration. Due to high surface energy, poorly dispersed nanoparticles form dense clusters that trap unhydrated cement cores and create internal micro-porosity, acting as artificial defect centers that increase matrix flaw size and initiate premature crack propagation [25], [26].

4. Cyclic Degradation, Hysteretic Damping, and Energy Dissipation

4.1. Hysteretic Loop Topology

In contrast to RC elements which exhibit severe pinching due to open flexural and shear crack planes, BF-ECC structural elements exhibit broad, robust, and stable hysteretic loops under severe drift ratios (up to 4.0–6.0%) [27]. The distributed microcracks (spacing < 3 mm, width < 25 micrometers) close rapidly upon load reversal, restoring structural compressive stiffness early in the reloading cycle [14].

4.2. Quantitative Hysteretic Damping Ratio Evaluation

Equivalent viscous damping ratio (h_e) quantifies the capacity to dissipate dynamic input energy [29]:

$$h_e = \frac{1}{2\pi} \frac{E_D}{E_{S0}} \quad (3)$$

Where ED is the total energy dissipated within one complete reversed cycle, and E_{S0} represents the peak elastic strain energy evaluated at maximum cycle displacement:

$$E_{S0} = 0.5 P_{\max} \Delta_{\max} \quad (4)$$

Components incorporating nano-silica modified BF-ECC sustain a linear-to-progressive energy absorption trajectory up to high drift demands, whereas standard RC components exhibit rapid fatigue of the hysteretic loop area [2], [30].

5. Critical Synthesis of Applications

The substitution of conventional concrete with NS/BF-ECC in plastic hinge zones eliminates cover spalling and provides intrinsic shear transfer through diagonal fiber-bridging mechanisms. For existing substandard buildings, thin BF-ECC jackets (25–50 mm) modified with nano-silica provide combined structural retrofitting and durability benefits, ensuring long-term resilience without extensive post-event repairs [32], [33].

6. Conclusions

- Engineered cementitious composites reinforced with basalt fibers (BF-ECC) demonstrate pseudo-strain hardening and saturated multiple microcracking.
- Colloidal nano-silica accelerates hydration kinetics and densifies the interfacial transition zone, significantly enhancing interfacial shear transfer and micro-crack bridging efficiency.
- Incorporating nano-silica modified BF-ECC mitigates hysteretic pinching, yielding equivalent viscous damping ratios between 15% and 24%.
- An optimum nano-silica threshold exists (typically 1.0–2.0 wt.%); dosages beyond this induce nanoparticle agglomeration and excessive matrix brittleness.

Nomenclature

Symbol	Definition
σ_{fc}	First crack tensile strength of the composite
$\sigma_{0\sigma0}$	Maximum complementary fiber bridging stress
$J_{b'Jb'}$	Complementary fiber bridging energy
$J_{tipJtip}$	Matrix crack-tip fracture energy
K_m	Matrix fracture toughness
E_m	Elastic modulus of the composite matrix
δ_0	Crack opening displacement at peak bridging stress
τ_0	Interfacial frictional shear bond strength
ED	Total dissipated hysteretic energy per cycle
E_{S0}	Peak elastic strain energy at maximum cycle displacement
h_e	Equivalent viscous hysteretic damping ratio
P_{max}	Peak lateral load sustained during cyclic excursion
Δ_{max}	Peak lateral displacement

Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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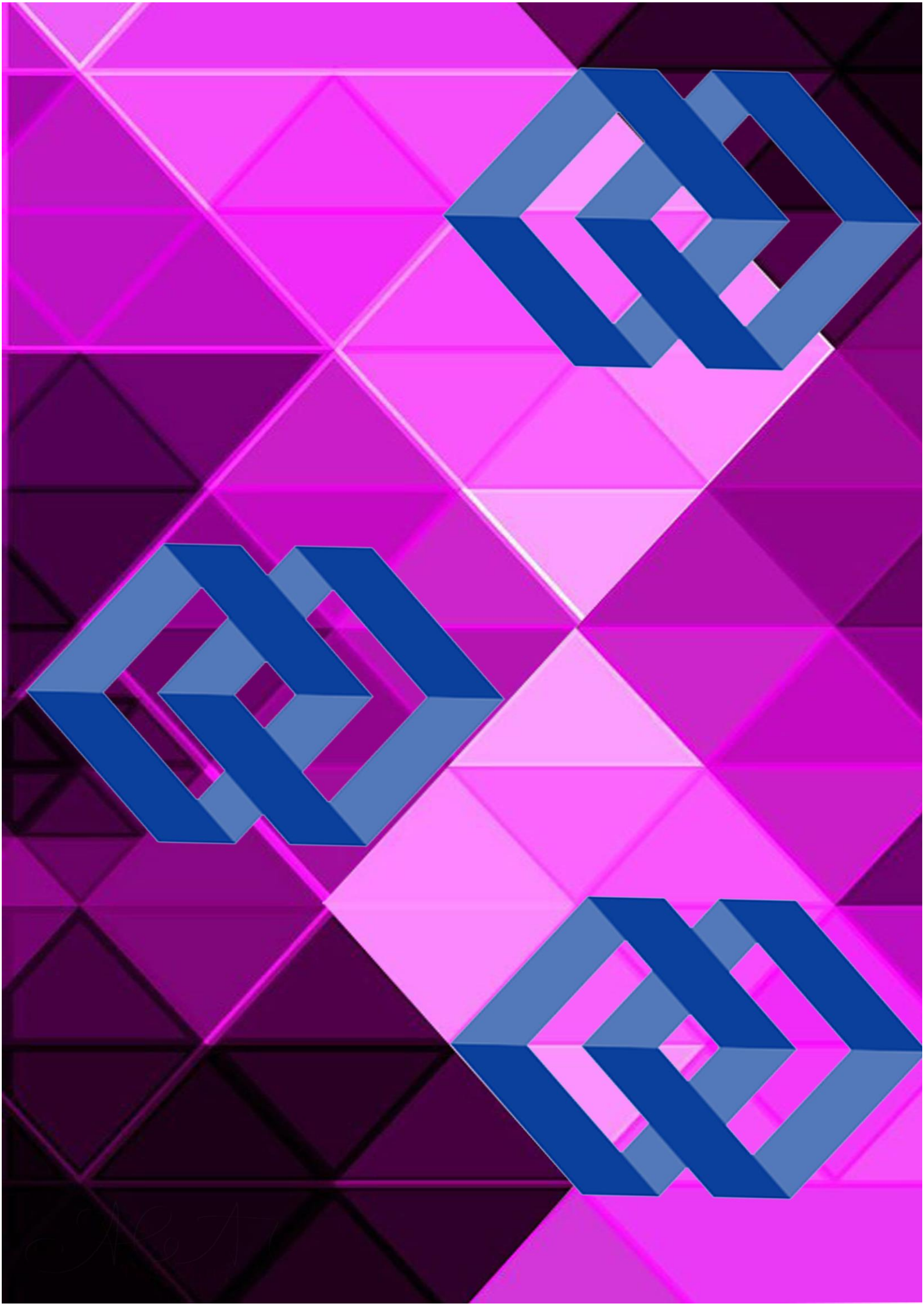
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