



applied sciences

Special Issue Reprint

New Trends in Long-Life Road Infrastructures

Materials and Structures, 2nd Edition

Edited by
Jue Li, Junhui Peng, Junfeng Gao and Wensheng Wang

mdpi.com/journal/applsci



New Trends in Long-Life Road Infrastructures: Materials and Structures, 2nd Edition

New Trends in Long-Life Road Infrastructures: Materials and Structures, 2nd Edition

Guest Editors

Jue Li

Junhui Peng

Junfeng Gao

Wensheng Wang



Basel • Beijing • Wuhan • Barcelona • Belgrade • Novi Sad • Cluj • Manchester

Guest Editors

Jue Li
College of Traffic &
Transportation
Chongqing Jiaotong
University
Chongqing
China

Junhui Peng
School of Traffic &
Transportation Engineering
Changsha University of
Science & Technology
Changsha
China

Junfeng Gao
Engineering Research Center
of Chongqing Education
Commission of China for
Resilient Pavement Materials
Chongqing Jiaotong
University
Chongqing
China

Wensheng Wang
College of Transportation
Jilin University
Changchun
China

Editorial Office

MDPI AG
Grosspeteranlage 5
4052 Basel, Switzerland

This is a reprint of the Special Issue, published open access by the journal *Applied Sciences* (ISSN 2076-3417), freely accessible at: https://www.mdpi.com/journal/applsci/special_issues/67654O3P54.

For citation purposes, cite each article independently as indicated on the article page online and as indicated below:

Lastname, A.A.; Lastname, B.B. Article Title. <i>Journal Name</i> Year , Volume Number, Page Range.
--

ISBN 978-3-7258-8669-2 (Hbk)

ISBN 978-3-7258-8670-8 (PDF)

<https://doi.org/10.3390/books978-3-7258-8670-8>

© 2026 by the authors. Articles in this reprint are Open Access and distributed under the Creative Commons Attribution (CC BY) license. The reprint as a whole is distributed by MDPI under the terms and conditions of the Creative Commons Attribution-NonCommercial-NoDerivs (CC BY-NC-ND) license (<https://creativecommons.org/licenses/by-nc-nd/4.0/>).

Contents

Preface	vii
Jue Li, Junhui Peng, Junfeng Gao and Wensheng Wang New Trends in Long-Life Road Infrastructures: Materials and Structures, 2nd Edition Reprinted from: <i>Appl. Sci.</i> 2026 , 16, 4127, https://doi.org/10.3390/app16094127	
	1
Bolin Jiang, Shanshan Wu, Feng Huang, Fuchuan Zhou and Xiaopeng Li Effect of Microbially Induced Calcium Carbonate Precipitation on Swelling and Performance Degradation of Expansive Soil Reprinted from: <i>Appl. Sci.</i> 2025 , 15, 6570, https://doi.org/10.3390/app15126570	
	7
Longni Wang, Hongyuan Fu, Qianfeng Gao, Jintao Luo, Jing Tang, Jianping Song, et al. Engineering Performance and Mechanism of Alkali-Activated Ground Granulated Blast Furnace Slag–Zeolite Powder Grouting Materials Reprinted from: <i>Appl. Sci.</i> 2025 , 15, 3345, https://doi.org/10.3390/app15063345	
	23
Xudong Zha, Chengzhi Wu, Runzhou Luo and Yaqiang Liu Optimization of Exposed Aggregate Concrete Mix Proportions for High Skid Resistance and Noise Reduction Performance Reprinted from: <i>Appl. Sci.</i> 2025 , 15, 5881, https://doi.org/10.3390/app15115881	
	42
Aiwu Yang, Tianli Liu, Hao Zhang and Boqu Zhang A Study on the Creep Characteristics of Gassy Clay Mixed with Silt Reprinted from: <i>Appl. Sci.</i> 2025 , 15, 5106, https://doi.org/10.3390/app15095106	
	61
Shuang Wang, Zhichen Wang, Ankang Yu, Huanan Yu, Zhongming He, Xiangzhu Meng, et al. Characterizing the Interaction Between Asphalt and Mineral Fillers in Hot Mix Asphalt Mixtures: A Micromechanical Approach Reprinted from: <i>Appl. Sci.</i> 2025 , 15, 2735, https://doi.org/10.3390/app15052735	
	76
Xize Han, Guangchen Sun, Helin Fu, Chao Tan, Zailong Huang, Peng Yin, et al. Experimental Study on Deformation and Damage Evolution of Cracked Red Sandstone Under Freeze–Thaw Cycles Reprinted from: <i>Appl. Sci.</i> 2024 , 14, 11174, https://doi.org/10.3390/app142311174	
	98
Jialing Song, Jiajin Leng, Jian Li, Hui Wei, Shangru Li and Feiyue Wang Application of Acoustic Emission Technique in Landslide Monitoring and Early Warning: A Review Reprinted from: <i>Appl. Sci.</i> 2025 , 15, 1663, https://doi.org/10.3390/app15031663	
	119
Bolin Jiang, Shanshan Wu, Qidong Xiong and Yongsheng Yao Enhancing Grout Filling Quality Assessment in Precast Concrete Sleeve Connections through a Collaborative Sensing Approach Reprinted from: <i>Appl. Sci.</i> 2024 , 14, 8932, https://doi.org/10.3390/app14198932	
	146
Haojun Liu, Xudong Zha and Yang Yin Risk Analysis of Service Slope Hazards for Highways in the Mountains Based on ISM-BN Reprinted from: <i>Appl. Sci.</i> 2025 , 15, 2975, https://doi.org/10.3390/app15062975	
	163
Xiaomin Dai, Guojin Su, Wei Tian and Long Cheng Research on the Operation, Maintenance, and Parameters of Expressway Mechanical and Electrical Equipment Based on Markov Prediction Reprinted from: <i>Appl. Sci.</i> 2024 , 14, 3628, https://doi.org/10.3390/app15073628	
	181

Zhibo Xu, Guihe Wang and Zhu Yang

Research on Construction Countermeasures for Freeze–Thaw Deformation of Permafrost Subgrade in Forest Regions of Northeast China

Reprinted from: *Appl. Sci.* **2025**, *15*, 12810, <https://doi.org/10.3390/app152312810> **202**

Wei Wang, Wei Li, Pengxiang Zhang and Lulu Liu

Investigation of Seepage Behavior and Settlement Deformation Mechanisms in Loess Embankment Foundation Systems in Eastern Gansu Province

Reprinted from: *Appl. Sci.* **2025**, *15*, 3789, <https://doi.org/10.3390/app15073789> **218**

Adham Mohammed Alnadish, Madhusudhan Bangalore Ramu, Abdullah O. Baarimah and Aawag Mohsen Alawag

Development of Enhanced Stress Prediction Models for Fixed Traffic Loads on Flexible Pavements Based on Response Surface Methodology (RSM) and Machine Learning (ML) Techniques

Reprinted from: *Appl. Sci.* **2025**, *15*, 1623, <https://doi.org/10.3390/app15031623> **233**

Kornel Nagy, Bernadett Bringye and Zoltan Karoly Lakner

Advancing Efficiency and Sustainability in Road Construction: A Bibliometric Review of Recent Innovations and Challenges

Reprinted from: *Appl. Sci.* **2026**, *16*, 1205, <https://doi.org/10.3390/app16031205> **256**

Preface

Road infrastructure is a fundamental component of modern transportation systems and plays a critical role in economic development, social connectivity, and public safety. However, many road networks are now exposed to multiple interacting challenges, including structural aging, increasing traffic loads, extreme climatic events, material degradation, slope instability, and rising demands for sustainable construction and maintenance. These challenges require new scientific understanding and more effective engineering solutions to improve the service life, resilience, and long-term performance of road infrastructure.

This Reprint, *New Trends in Long-Life Road Infrastructures: Materials and Structures*, 2nd Edition, brings together recent research findings on the materials, structural systems, monitoring methods, and maintenance strategies that support the development of long-life road infrastructures. Its scope covers sustainable and high-performance construction materials, soil and rock improvement, pavement and subgrade behavior, freeze thaw damage, slope hazard assessment, intelligent sensing, risk analysis, machine learning-based prediction, and optimized maintenance management. By combining research from materials science, geotechnical engineering, pavement engineering, structural engineering, and infrastructure management, this Reprint reflects the interdisciplinary nature of current road infrastructure research.

The purpose of this Reprint is to provide readers with an updated overview of scientific progress and practical technologies that can contribute to safer, more durable, and more sustainable road systems. The collected studies show that the extension of road infrastructure service life cannot rely on a single method. Instead, it requires coordinated advances in material design, structural analysis, environmental adaptation, real-time monitoring, data-driven decision making, and life cycle base maintenance. The papers included in this Reprint therefore offer both theoretical insights and engineering applications.

The motivation for preparing this Reprint arises from the urgent need to move road infrastructure development from reactive repair toward predictive, resilient, and sustainable management. Innovative materials can improve durability and reduce environmental impacts. Intelligent monitoring technologies can identify early signs of deterioration and support timely intervention. Advanced numerical models and data-driven approaches can improve performance prediction and risk assessment. Maintenance optimization methods can help infrastructure managers allocate resources more effectively and reduce life cycle costs.

This Reprint is addressed to researchers, graduate students, practicing engineers, infrastructure owners, maintenance agencies, and decision makers working in the fields of road engineering, pavement materials, geotechnical engineering, transportation infrastructure, and sustainable construction. We hope that this collection will provide useful references for future research and practical implementation, and that it will encourage further collaboration toward the construction and management of durable, resilient, and sustainable road infrastructure systems.

Jue Li, Junhui Peng, Junfeng Gao, and Wensheng Wang

Guest Editors

New Trends in Long-Life Road Infrastructures: Materials and Structures, 2nd Edition

Jue Li ^{1,*}, Junhui Peng ¹, Junfeng Gao ² and Wensheng Wang ³

¹ Key Laboratory of Highway Engineering of Ministry of Education, Changsha University of Science & Technology, Changsha 410114, China; pjh@csust.edu.cn

² College of Materials Science and Engineering, Chongqing Jiaotong University, Chongqing 400074, China; jfgao@cqjtu.edu.cn

³ College of Transportation, Jilin University, Changchun 130022, China; wangws@jlu.edu.cn

* Correspondence: lijue1207@126.com

1. Introduction

Global road infrastructure networks, which serve as the backbone of economic and social connectivity, are facing unprecedented challenges due to accelerated aging, intensifying climate change impacts, and increasing demands for sustainability and resilience [1]. The pursuit of extended service life, improved operational safety, and reduced lifecycle costs of road infrastructures has become a paramount objective for civil engineers and researchers globally. This Special Issue, “New Trends in Long-Life Road Infrastructures: Materials and Structures, 2nd Edition”, builds upon the success of its predecessor, presenting cutting-edge scientific advancements that address these critical challenges through interdisciplinary research encompassing materials science, structural engineering, geotechnics, and intelligent monitoring technologies.

The aging and deterioration of existing road infrastructure constitute a multifaceted global crisis [2]. In numerous regions, road networks constructed during periods of rapid economic expansion are now nearing the end of their design lives, necessitating substantial investments in maintenance, rehabilitation, and reconstruction [3]. For instance, China’s extensive highway network, an extraordinary engineering achievement, is currently contending with the cumulative effects of decades of heavy traffic and environmental exposure [4]. This degradation manifests in various forms, including pavement cracking and rutting, slope instability, damage from expansive soils, permafrost degradation, and failures in electromechanical systems. Collectively, these challenges pose significant threats to road safety and diminish operational efficiency.

Climate change exacerbates these vulnerabilities by increasing the frequency and intensity of extreme weather events [5]. Freeze–thaw cycles in the cold regions, intense rainfall-induced infiltration, prolonged drought–wet cycles, and rising temperatures directly degrade material properties and compromise structural stability, thereby accelerating road infrastructure deterioration [6]. These evolving environmental conditions necessitate a paradigm shift from traditional design approaches that assume static environmental conditions to resilient, adaptive infrastructures capable of withstanding dynamic climate stresses throughout their entire service lives [7].

Fortunately, these formidable challenges are accompanied by significant opportunities driven by rapid technological innovation. The traditional reactive, experience-based maintenance model is being transformed by a suite of emerging technologies [8]. Advanced numerical simulation tools facilitate precise modeling of complex multi-physics interactions, including thermo–hydro–mechanical coupling in permafrost regions. Data-driven

approaches, such as machine learning, Bayesian networks, and Markov chain models, offer powerful new methods for performance prediction, risk quantification, and optimized decision-making [9]. Novel sustainable materials, ranging from bio-inspired soil stabilizers to industrial byproduct-based geopolymers, provide pathways to enhance durability while minimizing environmental footprints [10,11]. Furthermore, intelligent monitoring systems that integrate wireless sensors, fiber optics, and remote sensing technologies enable real-time health assessments and early warnings, shifting maintenance strategies from scheduled to predictive and condition-based approaches.

This Special Issue compiles 14 high-quality contributions—12 original research articles and 2 comprehensive review papers—that highlight the latest advancements in long-life road infrastructure. These papers address key challenges across three interconnected domains: innovative materials for enhanced durability, smart monitoring and assessment technologies, and advanced design and maintenance strategies. Collectively, they illustrate how interdisciplinary research and technological innovation are propelling the transition toward more sustainable, resilient, and enduring road infrastructure systems.

2. An Overview of Published Articles

2.1. Materials Innovations for Extended Service Life

Materials innovation is essential for extending the service life of road infrastructure. This Special Issue includes six research articles that investigate novel materials and their engineering applications, focusing on durability, environmental sustainability, and performance optimization.

Jiang et al. (Contribution 1) examine the effectiveness of microbially induced calcium carbonate precipitation (MICP) technology in stabilizing expansive soils. Their study identifies 1 M as the optimal cementation solution concentration, resulting in a 391.67% increase in unconfined compressive strength and a 71% decrease in swell ratio. Additionally, wet–dry cycling experiments reveal that mechanical properties can degrade by up to 62.24% under cyclic conditions, providing valuable insights for the practical application of MICP in road construction.

Wang et al. (Contribution 2) develop an alkali-activated grouting material based on ground granulated blast furnace slag (GGBFS) and zeolite powder to overcome the limitations of traditional cement-based grouts. By adding calcium lignosulphonate as a retarding agent, they extend the initial setting time from 45 min to 2 h, improving workability. Their optimal mix ratio produces a grout with compressive strength comparable to silty mudstone, reducing interface stress concentration by 60%.

Zha et al. (Contribution 3) optimize the mix proportions of exposed aggregate concrete (EAC) to enhance skid resistance and noise reduction. Their findings indicate a 31.5% increase in pendulum value and a 6.48 dB reduction in tire–pavement noise compared to traditional grooved concrete, primarily in the mid-high frequency range.

Yang et al. (Contribution 4) investigate the creep characteristics of gassy clay mixed with silt, revealing the effects of confining pressure and gas content on shear deformation and long-term strength. They propose a parameter-efficient model for predicting long-term strength, contributing to marine infrastructure design.

Wang et al. (Contribution 5) utilize a micromechanical approach to analyze the interaction between asphalt and mineral fillers in hot mix asphalt mixtures. Their study identifies that higher concentrations of acidic SiO₂ weaken asphalt–filler interactions and highlights the influence of temperature and frequency on interaction strength.

Han et al. (Contribution 6) explore the deformation and damage evolution of cracked red sandstone under freeze–thaw cycles without external water replenishment. They find

that cumulative residual strain increases significantly with the number of cycles, providing important data for analyzing rock mass stability in cold regions.

2.2. Smart Monitoring and Assessment Technologies

Accurate monitoring of infrastructure condition is crucial for effective maintenance and extending service life. This Special Issue features four papers that advance smart monitoring and assessment technologies.

Song et al. (Contribution 7) provide a comprehensive review of acoustic emission (AE) techniques in landslide monitoring. They highlight the ability of AE to capture elastic wave signals generated by micro-damage in soil and rock masses, suggesting its superiority over conventional monitoring methods.

Jiang et al. (Contribution 8) propose a collaborative sensing approach that combines pre-embedded sensor methods and impact-echo techniques for assessing grout filling quality in precast concrete sleeve connections. Their method achieves field measurement deviations of less than 5%.

Liu et al. (Contribution 9) develop an integrated approach using interpretive structural modeling and Bayesian networks for quantitative risk analysis of slope hazards in mountainous highways. Their case study reveals key risk factors and provides a scientific basis for slope risk assessment.

Dai et al. (Contribution 10) apply multi-state Markov chain theory to predict failures and optimize maintenance strategies for highway electromechanical equipment. Their optimized maintenance strategy increases the long-term operational probability of critical devices to 71.31%.

2.3. Advanced Design and Maintenance Strategies

Integrating innovative materials and smart monitoring technologies into advanced structural design and maintenance management is essential for achieving long-life road infrastructure. This Special Issue includes three papers presenting novel design and maintenance optimization strategies.

Xu et al. (Contribution 11) investigate the freeze–thaw deformation of permafrost subgrade in Northeast China. They establish a thermo–hydro–mechanical coupling numerical model and propose a “two-step” construction countermeasure, limiting subgrade settlement to 23.5 mm over 20 years.

Wang et al. (Contribution 12) study seepage behavior and settlement mechanisms in loess embankment foundations in Eastern Gansu Province. Their research reveals moisture content variation patterns and a lag effect in regard to settlement, providing insights into slope stability.

Alnadish et al. (Contribution 13) develop enhanced stress prediction models for flexible pavements using response surface methodology and machine learning techniques. Their findings indicate that machine learning models outperform traditional models, offering valuable insights for pavement design and maintenance.

2.4. Bibliometric Review

Nagy et al. (Contribution 14) present a bibliometric review of innovations and challenges in road construction, focusing on efficiency and sustainability. They analyze global research trends, identifying key themes such as new materials and construction technology optimization, while emphasizing the need for adaptable solutions that balance performance, sustainability, and economic viability.

3. Conclusions

The 14 papers published in this Special Issue, “New Trends in Long-Life Road Infrastructures: Materials and Structures, 2nd Edition”, highlight significant advancements across three critical domains: materials innovation, smart monitoring, and advanced design and maintenance. Collectively, these contributions demonstrate how interdisciplinary research and technological innovation are tackling the pressing challenges posed by aging road infrastructure and climate change, thereby promoting the transition toward more sustainable, resilient, and durable road engineering.

Key contributions include the development of novel sustainable materials, such as MICP-stabilized soils and alkali-activated geopolymers, which offer enhanced durability and reduced environmental impacts compared to traditional materials. The Special Issue also showcases advancements in smart monitoring technologies, including collaborative sensing approaches for grout quality assessment, acoustic emission-based landslide early warning systems, and data-driven risk analysis frameworks. Furthermore, it presents innovative design and maintenance strategies, such as the two-step construction method for permafrost subgrade and machine learning-enhanced pavement stress prediction models, which optimize performance and extend the service life of road infrastructures.

These research findings provide valuable scientific insights and practical solutions for road engineers and infrastructure managers while also identifying important directions for future research. Future work should focus on further improving the long-term durability of innovative materials under extreme environmental conditions, developing more integrated and intelligent monitoring systems that combine multiple sensing modalities, and advancing digital twin technologies for real-time infrastructure management. Additionally, further research on the lifecycle assessment and economic viability of new technologies is essential to facilitate their widespread adoption in practice.

We extend our sincere gratitude to all the authors who contributed their valuable research to this Special Issue, to the reviewers for their rigorous and constructive feedback, and to the editorial team of Applied Sciences for their excellent support throughout the publication process. We hope this collection of papers will inspire further research and innovation in the field of long-life road infrastructure, contributing to the development of more sustainable and resilient transportation systems worldwide.

Author Contributions: Conceptualization, J.L., J.P., J.G. and W.W.; writing—original draft preparation, J.L., J.P., J.G. and W.W.; and review and editing, J.L. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Acknowledgments: The editors extend their sincere gratitude to the authors for their significant contributions and valuable research presented in this Special Issue. We also appreciate the support of the *Applied Sciences* Editorial Office throughout the review and publication process.

Conflicts of Interest: The authors declare no conflicts of interest.

List of Contributions:

1. Jiang, B.; Wu, S.; Huang, F.; Zhou, F.; Li, X. Effect of Microbially Induced Calcium Carbonate Precipitation on Swelling and Performance Degradation of Expansive Soil. *Appl. Sci.* **2025**, *15*, 6570.
2. Wang, L.; Fu, H.; Gao, Q.; Luo, J.; Tang, J.; Song, J.; Li, Y.; Yu, G. Engineering Performance and Mechanism of Alkali-Activated Ground Granulated Blast Furnace Slag–Zeolite Powder Grouting Materials. *Appl. Sci.* **2025**, *15*, 3345.
3. Zha, X.; Wu, C.; Luo, R.; Liu, Y. Optimization of Exposed Aggregate Concrete Mix Proportions for High Skid Resistance and Noise Reduction Performance. *Appl. Sci.* **2025**, *15*, 5881.

4. Yang, A.; Liu, T.; Zhang, H.; Zhang, B. A Study on the Creep Characteristics of Gassy Clay Mixed with Silt. *Appl. Sci.* **2025**, *15*, 5106.
5. Wang, S.; Wang, Z.; Yu, A.; Yu, H.; He, Z.; Meng, X.; Gong, Z.; Guan, D.; Zhang, F. Characterizing the Interaction Between Asphalt and Mineral Fillers in Hot Mix Asphalt Mixtures: A Micromechanical Approach. *Appl. Sci.* **2025**, *15*, 2735.
6. Han, X.; Sun, G.; Fu, H.; Tan, C.; Huang, Z.; Yin, P.; Zhang, Q.; Fan, W.; Yin, S. Experimental Study on Deformation and Damage Evolution of Cracked Red Sandstone Under Freeze–Thaw Cycles. *Appl. Sci.* **2024**, *14*, 11174.
7. Song, J.; Leng, J.; Li, J.; Wei, H.; Li, S.; Wang, F. Application of Acoustic Emission Technique in Landslide Monitoring and Early Warning: A Review. *Appl. Sci.* **2025**, *15*, 1663.
8. Jiang, B.; Wu, S.; Xiong, Q.; Yao, Y. Enhancing Grout Filling Quality Assessment in Precast Concrete Sleeve Connections through a Collaborative Sensing Approach. *Appl. Sci.* **2024**, *14*, 8932.
9. Liu, H.; Zha, X.; Yin, Y. Risk Analysis of Service Slope Hazards for Highways in the Mountains Based on ISM-BN. *Appl. Sci.* **2025**, *15*, 2975.
10. Dai, X.; Su, G.; Tian, W.; Cheng, L. Research on the Operation, Maintenance, and Parameters of Expressway Mechanical and Electrical Equipment Based on Markov Prediction. *Appl. Sci.* **2025**, *15*, 3628.
11. Xu, Z.; Wang, G.; Yang, Z. Research on Construction Countermeasures for Freeze–Thaw Deformation of Permafrost Subgrade in Forest Regions of Northeast China. *Appl. Sci.* **2025**, *15*, 12810.
12. Wang, W.; Li, W.; Zhang, P.; Liu, L. Investigation of Seepage Behavior and Settlement Deformation Mechanisms in Loess Embankment Foundation Systems in Eastern Gansu Province. *Appl. Sci.* **2025**, *15*, 3789.
13. Alnadish, A.M.; Ramu, M.B.; Baarimah, A.O.; Alawag, A.M. Development of Enhanced Stress Prediction Models for Fixed Traffic Loads on Flexible Pavements Based on Response Surface Methodology (RSM) and Machine Learning (ML) Techniques. *Appl. Sci.* **2025**, *15*, 1623.
14. Nagy, K.; Bringye, B.; Lakner, Z.K. Advancing Efficiency and Sustainability in Road Construction: A Bibliometric Review of Recent Innovations and Challenges. *Appl. Sci.* **2026**, *16*, 1205.

References

1. Gössling, S.; Neger, C.; Steiger, R.; Bell, R. Weather, climate change, and transport: A review. *Nat. Hazards* **2023**, *118*, 1341–1360. [CrossRef]
2. Capacci, L.; Biondini, F.; Frangopol, D.M. Resilience of aging structures and infrastructure systems with emphasis on seismic resilience of bridges and road networks. *Resilient Cities Struct.* **2022**, *1*, 23–41. [CrossRef]
3. Mikolaj, J.; Remek, L.; Kozel, M.; Šedivý, Š. Maximizing Pavement Service Life: A Comprehensive Process Model Based on Structural Life Extension, Serviceability Deterioration Processes, and Asset Value. *Appl. Sci.* **2025**, *15*, 8782. [CrossRef]
4. Cao, L.; Li, L.; Yang, C.; Zhang, B.; Dong, Z. Performance prediction of expressway pavement in high maintenance level areas based on cosine deterioration equation: A case study of Zhejiang Province in China. *J. Road Eng.* **2022**, *2*, 267–278. [CrossRef]
5. Sias, J.E.; Dave, E.V.; Underwood, B.S.; Bowers, B.F.; Harvey, J.T.; Henning, T.F.; Tighe, S.L.; Jacobs, J.M.; Pregnolato, M.; Qiao, Y. Climate change impacts on roadways. *Nat. Rev. Earth Environ.* **2025**, *6*, 555–573. [CrossRef]
6. Zhao, K.; Dong, Z.; Ma, X.; Wang, W. Analyzing the Freeze–Thaw Cycle Zoning Patterns of Asphalt Pavements with Laboratory Split Tensile Experiments and the k-Means Clustering Methodology: A Case Study of China. *J. Cold Reg. Eng.* **2026**, *40*, 04025055. [CrossRef]
7. Mendoza-Sanchez, J.F.; Alonso-Guzman, E.M.; Martinez-Molina, W.; Chavez-Garcia, H.L.; Soto-Espitia, R.; Delgado-Alamilla, H.; Obregon-Biosca, S.A. A Critical Review of Pavement Design Methods Based on a Climate Approach. *Sustainability* **2024**, *16*, 7211. [CrossRef]
8. Dong, S.; Tong, Y.; Wan, Z.; Zuo, C.; Luo, J.; Cui, Z.; Wang, H.; Dai, Z. Multi-objective maintenance and rehabilitation decision-making modelling for highway networks: Balancing economy, environment, and maintenance benefits. *Transp. A Transp. Sci.* **2025**, 2507898. [CrossRef]
9. Tamagusko, T.; Gomes Correia, M.; Ferreira, A. Machine learning applications in road pavement management: A review, challenges and future directions. *Infrastructures* **2024**, *9*, 213. [CrossRef]

10. Li, J.; Hou, Q.; Peng, J.; Hu, H.; Vizzari, D. A critical review of heavy metal characteristics and risk assessment approaches of municipal solid waste incineration bottom ash in subgrade construction. *J. Traffic Transp. Eng. (Engl. Ed.)* **2025**, *12*, 1150–1173. [CrossRef]
11. Li, H.; Shi, L.; Li, J.; Ma, T.; Tan, Y.; Chen, W.; Lin, Y. Durability influencing factors and attenuation characteristics of composite modified asphalt anti-skid surface layer. *Constr. Build. Mater.* **2026**, *516*, 145631. [CrossRef]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Effect of Microbially Induced Calcium Carbonate Precipitation on Swelling and Performance Degradation of Expansive Soil

Bolin Jiang ^{1,2}, Shanshan Wu ^{2,3}, Feng Huang ^{2,*}, Fuchuan Zhou ¹ and Xiaopeng Li ²

¹ Department of Science and Technology, Chongqing Vocational Institute of Engineering, Chongqing 402260, China; bolinjiang@cqvie.edu.cn (B.J.); zfcot150@cqvie.edu.cn (F.Z.)

² School of Civil Engineering, Chongqing Jiaotong University, Chongqing 400074, China; shanshanwu84@163.com (S.W.); xiaopengli85@163.com (X.L.)

³ School of Railway and Architecture, Chongqing Vocational College of Public Transportation, Chongqing 402247, China

* Correspondence: huangfeng216@126.com

Abstract: Expansive soils exhibit pronounced behaviors of swelling, shrinkage, and over-consolidation, leading to significant engineering challenges. To address these issues, this study investigated the influence of different cementation solution concentrations on the swelling and mechanical properties of microbially induced calcium carbonate precipitation (MICP)-treated expansive soils. Additionally, a series of wet–dry cycling experiments were conducted to analyze the performance degradation of the MICP-treated soils under cyclic environmental conditions. The results showed that as the cementation solution concentration increased, the free swell ratio and unconfined swell ratio of the soil samples decreased by 39.29% and 71%, respectively. The unconfined compressive strength, cohesion, and internal friction angle of the MICP-treated soils also exhibited significant improvements, with increases of 391.67%, 33.76%, and 91.67%, respectively. A cementation solution concentration of 1 M was found to be the optimal for achieving substantial enhancements in the soil properties. However, under continuous wet–dry cycling, the mechanical properties of the MICP-treated soils experienced degradation, with the unconfined compressive strength decreasing by up to 62.24%. These findings highlight the potential of MICP as an effective and eco-friendly technique for stabilizing expansive soils.

Keywords: expansive soil; microbially induced calcium carbonate precipitation (MICP); swelling characteristics; mechanical properties; wet–dry cycling

1. Introduction

Expansive soils, notably characterized by their high contents of montmorillonite and illite minerals, exhibit pronounced behaviors of swelling, shrinkage, and over-consolidation [1]. These properties can lead to significant engineering challenges, including uneven settlement, slope instability, and pavement heaving [2]. These detrimental effects have led to the metaphorical characterization of expansive soils as the “cancer of highways” [3]. The extensive distribution of these soils, coupled with the often concealed and long-term nature of the associated issues, results in global annual repair costs amounting to billions of dollars [4]. Consequently, the development of effective soil improvement techniques is essential for the rational and sustainable utilization of expansive soils.

To address the challenges posed by expansive soils, researchers have extensively investigated a variety of physical and chemical stabilization techniques [5]. Physical

stabilization methods typically involve the incorporation of non-expansive fillers, such as fine sand, crushed stone, or fibers, to reduce the montmorillonite and illite mineral contents, thereby suppressing the swelling and shrinkage behavior while simultaneously improving mechanical properties [6,7]. For instance, Huang et al. [8] demonstrated the effectiveness of basalt fiber in enhancing the stability of expansive soils. Their findings indicated that the addition of 4% basalt fiber significantly mitigated swelling and shrinkage while also enhancing the mechanical properties of the soil. Similarly, Blayi et al. [9] reported that the inclusion of rock powder, fine sand, and gravel improved both the stability and mechanical performance of expansive soils. However, it is important to note that the effectiveness of physical stabilization is somewhat limited and is primarily suitable for soils with low expansiveness.

In contrast, chemical stabilization approaches focus on modifying the internal particle structure of expansive soils or inducing gel-like precipitations to enhance their swelling and shrinkage characteristics [10,11]. The previous studies [12,13] have shown that the addition of lime and cement positively influences the engineering properties and swelling–shrinkage behavior of expansive soils. For instance, Mohamed et al. [14] effectively improved the swelling–shrinkage properties and enhanced the compressive and shear strength of expansive soils through various mixtures, including 6% lime with 12% zeolite, 1% cement with 10% fly ash, and sawdust ash. Furthermore, Zhou et al. [15] identified an optimal improvement in expansive soils with the incorporation of 0.6% cationic polyacrylamide. While these chemical methods can effectively reduce swelling and shrinkage, they often raise concerns regarding energy consumption and environmental pollution [16].

In recent years, microbially induced calcium carbonate precipitation (MICP) has emerged as a promising and eco-friendly technique for soil improvement [17]. MICP leverages the interaction between specific microorganisms and expansive soils to facilitate the formation of calcium carbonate precipitates, which subsequently alters the internal structure and mineral composition of the soil [18]. Numerous studies have demonstrated the effectiveness of MICP in enhancing various properties of expansive soils, including material properties, swelling rates, and mechanical characteristics [19,20]. For instance, Osinubi et al. [21] utilized MICP technology to treat the natural soil, resulting in significant improvements in key properties such as Atterberg limits and linear shrinkage, with optimal performance being achieved at a specific bacteria-to-cementation reagent ratio. Similarly, Chittoori et al. [22] reported that MICP treatment led to substantial increases in both the compressive and shear strength of expansive soils. These findings underscore the potential of MICP as a viable solution for mitigating the challenges posed by expansive soil behavior [23]. However, certain environmental conditions can adversely affect the performance of MICP-treated soils. In regions with hot and humid climates, such as Guangxi and Yunnan provinces in China, cyclic wet–dry conditions can lead to a decline in the bearing capacity of MICP-treated expansive soil bases [24]. This phenomenon poses challenges for the long-term stability and effectiveness of MICP applications in these regions [25]. Additionally, it is important to note that the natural concentration of calcium ions in expansive soils is often low, necessitating the addition of an appropriate calcium source to enhance the production of calcium carbonate precipitates [26]. Despite the promising results of MICP, there is currently limited research focused on determining the optimal concentration of the cementation solution required for the effective MICP treatment of expansive soils.

This study aimed to investigate the influence of different cementation solution concentrations on the physical, swelling, and mechanical properties of MICP-treated expansive soils. Furthermore, a series of wet–dry cycling experiments were conducted to analyze the performance degradation of the MICP-treated expansive soils under cyclic

environmental conditions. The findings of this study contribute to the development of sustainable and eco-friendly soil stabilization techniques for expansive soil-prone regions, providing insights into the long-term performance of MICP-treated soils under varying environmental conditions.

2. Materials and Methods

2.1. Materials

The expansive soil samples used in this study were collected from a construction site in Guangxi province, China. Surface soils exhibiting potential rainfall-induced weathering were systematically excluded, with all test specimens extracted from the primary expansive clay layer (1.5–2.0 m depth). The soil samples were yellowish-brown in color and had a relatively high natural moisture content (27.3%). The physical properties and swelling potential of the expansive soil were determined, as presented in Tables 1 and 2.

Table 1. Physical properties of the expansive soil.

Physical Properties	Maximum Dry Density (g/cm ³)	Optimal Moisture Content (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index
Value	1.68	21.90	32.83	57.50	24.67
Method	ASTM D698 [27]	ASTM D698 [27]	ASTM D4318 [28]	ASTM D4318 [28]	ASTM D4318 [28]

Table 2. Swelling potential of the expansive soil.

Free Swell Rate (%)	Unconfined Swell Rate (%)	Confined Swell Rate (%)	Method
56	16.38	7.19	ASTM D4546(B) [29]

The MICP process was facilitated by *Sporosarcina pasteurii* (ATCC11859), a common ureolytic bacteria widely used in MICP applications. The bacterial culture was activated and cultivated on Petri dishes prior to the experiments. The cultivation conditions for the bacterial strain are presented in Table 3. After the cultivation was completed, the optical density (OD₆₀₀) of the bacterial suspension was measured, and the suspension was considered ready for use when the OD₆₀₀ value reached 1.0. The bacterial suspension was then stored at 4 °C to avoid potential mutations from repeated subculturing.

Table 3. Cultivation conditions for *Sporosarcina pasteurii*.

Beef Extract	Peptone	Urea	NaCl	Rotational Speed	Incubation Time
3 g/L	10 g/L	60.06 g/L	5 g/L	180 rpm	7 days

2.2. Sample Preparation

The expansive soil samples were first air-dried, crushed, and sieved through a 2 mm mesh to prepare the soil for the stabilization experiments. To mitigate the potential impact of insufficient natural calcium ion concentration in the expansive soil, cementation solutions with different concentrations were prepared. Specifically, five cementation solution concentrations were investigated: 0 M, 0.5 M, 0.75 M, 1.0 M, 1.25 M, and 1.5 M. The molar ratio of urea to calcium chloride (CaCl₂) was maintained at 1:1 in all the cementation solutions, as recommended in previous studies.

The MICP treatment was conducted using a mixing method. The bacterial suspension and the cementation solution were added to the expansive soil in a 1:1 ratio by mass and thoroughly mixed until the soil moisture content reached the target value. Based on previous studies that determined the optimal moisture content for maximizing compaction density and performance of expansive soils, all soil improvement tests in this work were consistently conducted at a fixed target moisture content of 21.9% to ensure experimental repeatability and result comparability.

The MICP-treated soil samples with different cementation solution concentrations (0 M, 0.5 M, 0.75 M, 1.0 M, 1.25 M, and 1.5 M) were labeled as S-0, S-1, S-2, S-3, S-4, and S-5, respectively. After the mixing, the MICP-treated soil samples were placed in a temperature- and humidity-controlled curing chamber (temperature of 25 °C and relative humidity of 95%) and cured for 7 days to allow for the completion of the MICP process.

The cured soil samples were then subjected to various physical, swelling, and mechanical property tests, as described in the following sections.

2.3. Test Methods

The overall test procedure, encompassing the preparation of soil samples, application of MICP treatment, and subsequent testing for swelling properties and mechanical properties, is systematically illustrated in Figure 1.

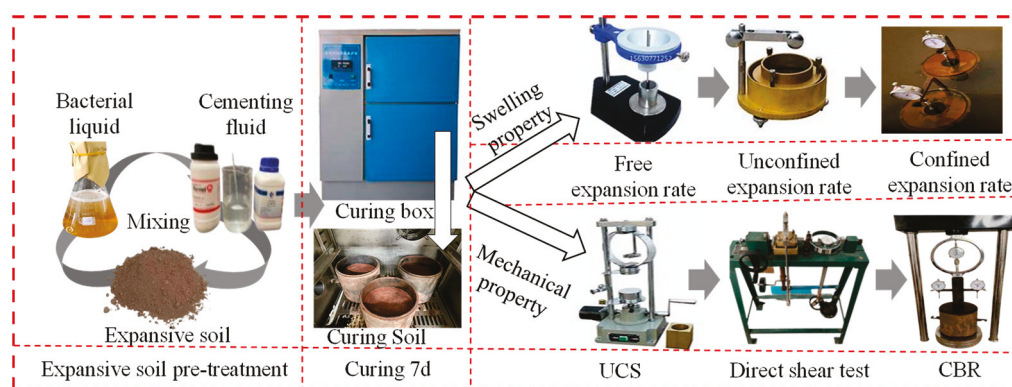


Figure 1. Experimental protocol flowchart.

All the swelling and mechanical property tests were performed on the MICP-treated expansive soil samples with different cementation solution concentrations, as well as the untreated control sample (S-0). The results were analyzed to evaluate the effectiveness of the MICP treatment and the influence of the cementation solution concentration on the soil properties. In addition, each experimental sample was set up with 3 parallel samples.

2.3.1. Swelling Property Tests

The swelling behavior of MICP-treated expansive soils was evaluated through free swelling and unconfined swelling tests performed in accordance with ASTM D4546(B) [29]. Meanwhile, the confined swelling property was measured during the CBR test, following ASTM D1883 [30].

The soil samples were placed in a free expansion instrument and consolidometer and saturated with water. The free and vertical swell deformation was recorded over time until no further expansion was observed, ensuring a thorough assessment of the free swelling and unconfined swelling potential.

2.3.2. Mechanical Property Tests

The mechanical properties of the MICP-treated expansive soil samples were evaluated through unconfined compressive strength (UCS), direct shear, and California bearing ratio (CBR) tests.

The UCS test was conducted in accordance with ASTM D2166 [31]. Cylindrical soil samples were prepared and tested under axial compression until failure. The maximum compressive strength was recorded as the UCS.

The shear strength of the MICP-treated expansive soil samples was evaluated through direct shear tests, following ASTM D3080 [32]. The specimens were placed in the direct shear apparatus and subjected to a series of normal stresses (50 kPa, 100 kPa, and 150 kPa) to determine the shear strength parameters, namely the cohesion (c) and the angle of internal friction (φ).

The CBR test was performed following the procedures outlined in ASTM D1883 [30]. Soil samples were compacted in a CBR mold and soaked in water for 96 h, and the confined swelling property was measured during the test. Subsequently, the samples were subjected to a penetration test. The CBR value was calculated as the ratio of the test load to the standard load, expressed as a percentage.

2.3.3. Scanning Electron Microscopy Test

In this study, a scanning electron microscopy (SEM) was employed to examine the microstructural variations between untreated expansive soil (S-0) and soil treated with a 1.0 M cementing solution (S-3). Standard soil specimens ($8 \times 8 \times 15 \text{ mm}^3$) were prepared and freeze-dried under liquid nitrogen vacuum to preserve their original pore structure. For SEM observation, samples were sputter-coated with metal and mounted on the specimen stage. Microstructural characterization was performed at 15 kV accelerating voltage and $50\times$ magnification. These observations effectively revealed the microscale mechanism of soil structure improvement through cementation.

2.3.4. Wet–Dry Cycling Tests

To analyze the performance degradation of the MICP-treated expansive soil under cyclic environmental conditions, a series of wet–dry cycling tests were conducted. The soil sample treated with a 1.0 M cementation solution (S-3) was selected for the wet–dry cycling tests, which is the optimal binder concentration determined by the swelling characteristics and mechanical performance tests of the improved soil. In addition, the untreated control sample (S-0) were selected as the control group.

The wet–dry cycling test procedure was as follows:

1. Prepare test specimens from the S-0 and S-3 soil sample.
2. Dry the soil specimens in a 60 °C oven for 23 h, followed by 1 h of resting in a 25 °C oven.
3. Submerge the soil specimens in water for 23 h, followed by 1 h of resting in a 25 °C oven.
4. Repeat steps 2 and 3 to complete 0, 3, 6, 9, 12, and 15 wet–dry cycles.
5. After each specified number of wet–dry cycles, measure the mass loss, volume deformation, UCS, and shear strength of the soil specimens.

The wet–dry cycling test was designed to simulate the cyclic environmental conditions, particularly the wet–dry cycles, experienced by the MICP-treated expansive soil in regions with hot and humid climates in China. The test results were used to evaluate the long-term performance and durability of the MICP-treated expansive soil under these challenging environmental conditions.

3. Results and Discuss

3.1. Swelling Characteristics of Treated Soils

The results of the free swell tests and confined swell tests for the soil samples are presented in Figures 2 and 3, respectively. These results reveal that as the cementation solution concentration increased, the free swell ratio of the soil samples decreased from 56% for S-0 to 34% for S-5, a reduction of 39.29%. Similarly, the unconfined swell ratio decreased from 16.38% for S-0 to 4.75% for S-5, a reduction of 71%.

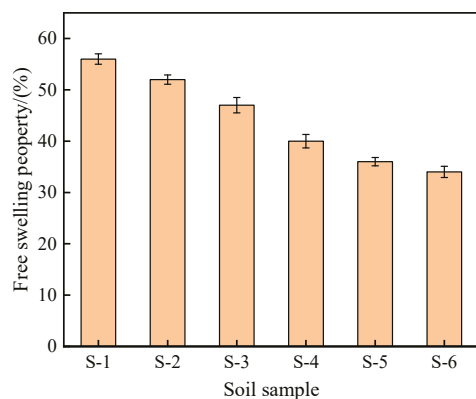


Figure 2. Free swell rate test results.

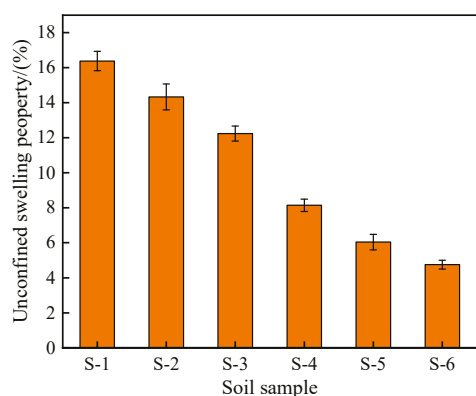


Figure 3. Unconfined swell ratio test results.

The free swell ratio and unconfined swell ratio both exhibited a decreasing trend, with a more rapid decline at lower cementation solution concentrations and a more gradual decline at higher concentrations. Specifically, for the soil samples S-0 to S-3 (cementation solution concentration less than 1 M), the free swell ratio and unconfined swell ratio decreased by 28.57% and 50.31%, respectively. In contrast, for the soil samples S-3 to S-5 (cementation solution concentration greater than 1 M), the decreases in the free swell ratio and confined swell ratio were only 10.72% and 20.69%, respectively.

These results suggest that a cementation solution concentration of 1 M is effective in reducing the swelling potential of the expansive soil. The reduction in swelling characteristics can be attributed to the precipitation of calcium carbonate induced by the *Sporosarcina pasteurii* bacteria and the cementation solution. The CaCO_3 precipitation fills the pore spaces and alters the internal particle structure of the expansive soil, effectively improving its water-induced swelling behavior.

Moreover, the significant decrease in swelling ratios, especially at lower cementation solution concentrations, indicates that the MICP treatment enhances the soil's resistance

to volume changes upon wetting. This improvement is crucial for mitigating the adverse effects of swelling in expansive soils, such as uneven settlement and structural failures.

3.2. Mechanical Properties of Treated Soils

The UCS test results for the S-0 to S-5 soil samples are shown in Figure 4. As the cementation solution concentration increased, the UCS increased from 0.24 MPa to 1.18 MPa, an increase of 391.67%. The rate of change in the UCS exhibited a rapid increase at first, followed by a slower increase. The S-3 soil sample, corresponding to a cementation solution concentration of 1 M, was the turning point for the change in the compaction characteristics of the MICP-treated expansive soil. When the cementation solution concentration was less than 1 M (S-0 to S-3), the UCS increased from 0.24 MPa to 0.96 MPa, an increase of 300%. When the cementation solution concentration was greater than 1 M (S-3 to S-5), the UCS increased from 0.96 MPa to 1.18 MPa, an increase of 91.67%.

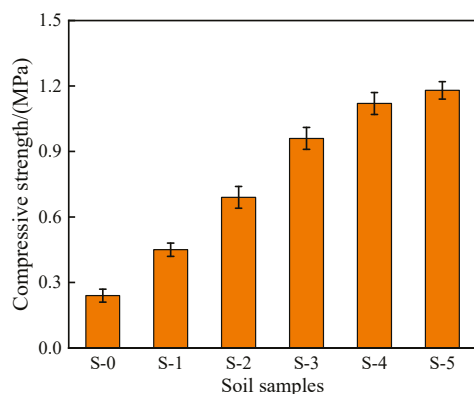


Figure 4. Compression test results.

The direct shear test results for the S-0 to S-5 soil samples are shown in Figure 5. As the cementation solution concentration increased, the cohesion increased from 20.67 kPa to 27.65 kPa, an increase of 33.76%, and the internal friction angle increased from 18.53° to 35.52° , an increase of 91.67%. The rate of change in the cohesion and internal friction angle exhibited a rapid increase at first, followed by a slower increase. The S-3 soil sample, corresponding to a cementation solution concentration of 1 M, was the turning point for the change in the compaction characteristics of the MICP-treated expansive soil. When the cementation solution concentration was less than 1 M (S-0 to S-3), the increases in cohesion and the internal friction angle were 17.77% and 58.33%, respectively. When the cementation solution concentration was greater than 1 M (S-3 to S-5), the increases in cohesion and the internal friction angle were 15.99% and 33.34%, respectively.

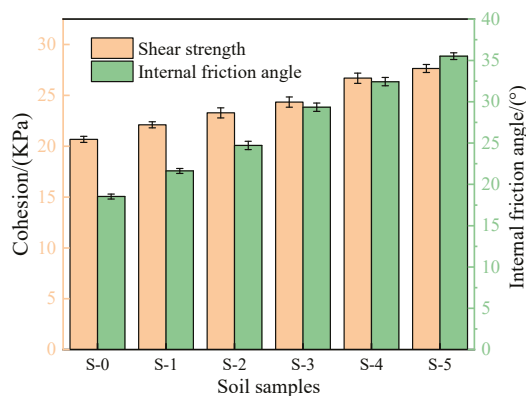


Figure 5. Shear test results.

The CBR test results for the S-0 to S-5 soil samples are shown in Figures 6 and 7. The CBR value increased from 2.05% to 10.33%, an increase of 403.90%. A minimum value point occurred at a cementation solution concentration of 1 M, with the S-3 soil sample having a CBR value of 8.56%, which already meets the requirement for the highway subgrade filler CBR value in China. The load expansion rate results show that the S-1 to S-5 soil samples had a significantly lower load expansion rate compared to the S-0 soil sample, with a reduction of ranging from 13.40% to 61.44%. The overall change rate of the load expansion rate for the S-0 to S-5 soil samples exhibited a rapid decrease at first, followed by a slower decrease. When the cementation solution concentration was greater than 1 M, the decrease in the load expansion rate slowed down significantly.

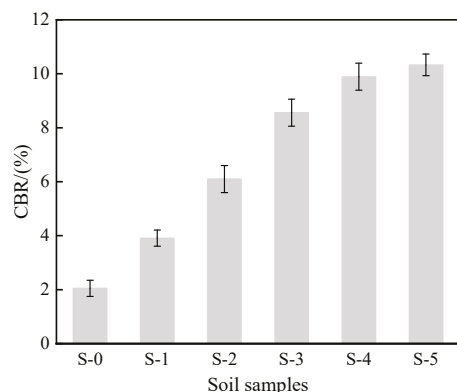


Figure 6. CBR value results.

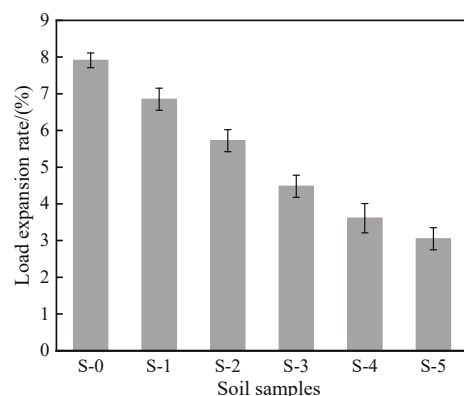


Figure 7. Load expansion rate test results.

These results indicate that the mechanical properties of the expansive soil were significantly improved after MICP treatment. This is likely due to the formation of calcium carbonate precipitates through the interaction between *Sporosarcina pasteurii*, the cementation solution, and the expansive soil. Some of the precipitates were adsorbed onto the surrounding fine particles, forming aggregates, while others filled the pores between the soil particles [33]. This resulted in a more compact internal structure, with reduced voids and increased particle interlocking and frictional effects, allowing the improved soil to withstand higher stresses.

Furthermore, the substantial increase in UCS, cohesion, and the internal friction angle, along with the significant reduction in the load expansion rate, highlights the effectiveness of MICP treatment in enhancing the load-bearing capacity and stability of expansive soils. The findings suggest that a cementation solution concentration of 1 M is optimal for achieving significant mechanical property improvements.

3.3. Discussion

The application of microbially induced carbonate precipitation (MICP) technology demonstrates significant potential for improving the expansive characteristics and mechanical properties of expansive soils. This finding aligns with previous studies which reported substantial enhancements in soil mechanical parameters through microbial treatment. For instance, Tian et al. [34] observed a 33.5% increase in soil cohesion (from 29.52 kPa to 39.41 kPa) and a 46.94% improvement in the internal friction angle (from 20.13° to 29.58°). Similarly, Chittoori and Neupane [20] documented a 252% enhancement in unconfined compressive strength (UCS), with values increasing from 0.25 MPa to 0.88 MPa.

Notably, some discrepancies exist between the improvement levels reported in these studies and our current findings. These variations may be attributed to differences in the mineralogical composition of expansive soils across geographical regions.

Furthermore, it is important to recognize that the efficacy of microbial soil modification can be influenced by multiple factors, including the bacterial solution concentration and curing conditions (temperature, duration, and pH) [35]. While the present study primarily examines the effect of cementation solution concentration while maintaining constant bacterial solution concentrations and curing parameters, future investigations should systematically evaluate the impacts of these additional variables to optimize the MICP treatment process.

3.4. Microscopic Mechanism of MICP-Treated Expansive Soil

The MICP treatment was further investigated at the microscopic level to understand the mechanism by which it improves the properties of expansive soil. Scanning electron microscopy (SEM) was employed to examine the microstructural changes in the untreated soil (S-0) and the soil treated with a 1.0 M cementation solution (S-3).

Figure 8a presents the SEM images of the untreated soil sample (S-0). The soil particles exhibit a dispersed and block structure, characteristic of expansive soils. The pore spaces between the particles are relatively large, allowing for the penetration of water and the subsequent swelling of the soil. In contrast, the SEM images of the MICP-treated soil sample (S-3) shown in Figure 8b reveal a markedly different microstructure. The soil particles are bound together by the precipitation of calcium carbonate, forming a denser and more agglomerated structure. The pore spaces between the particles are significantly reduced, with the CaCO_3 filling the voids and cementing the soil grains together. CaCO_3 enhances the soil stability by binding particles together and adsorbing onto expansive soil surfaces while also modifying their contact behavior.

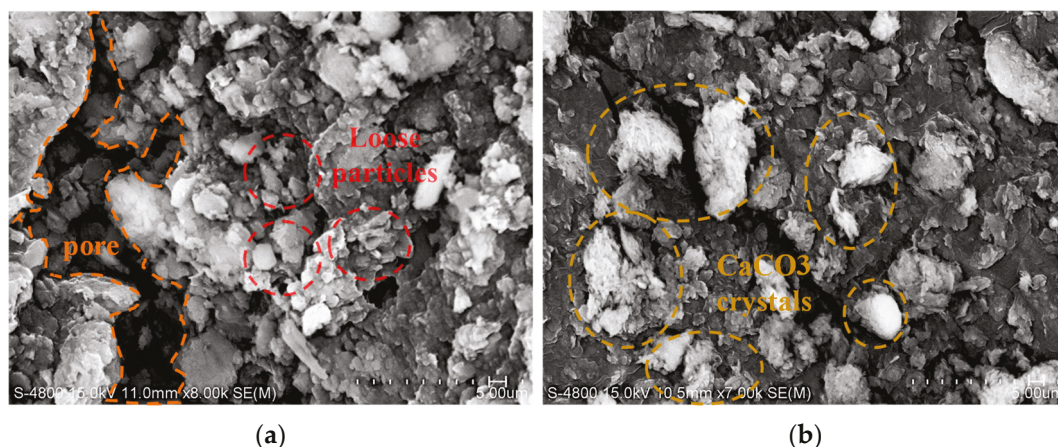


Figure 8. SEM images of (a) the untreated soil sample (S-0); (b) the MICP-treated soil sample (S-3).

The CaCO_3 precipitation has a twofold effect on the expansive soil properties. First, the cementation of the soil particles reduces the overall porosity and permeability of the soil, limiting the ingress of water and the subsequent swelling of the soil. Second, the CaCO_3 precipitates act as a bonding agent, improving the overall strength and stability of the soil structure. Therefore, the microscopic evidence provided by the SEM analysis supports the observed improvements in the swelling and mechanical properties of the MICP-treated expansive soil, as discussed in the previous sections.

4. Performance Degradation of MICP-Treated Soil Under Wet–Dry Cycling

4.1. Degradation of Mechanical Properties

The results of the UCS tests conducted on the specimens after various wet–dry cycling intervals are presented in Figure 9. The results show that the UCS of the MICP-treated soil samples gradually decreased with an increasing number of wet–dry cycles, exhibiting a trend of a rapid initial decline followed by a more gradual reduction. After nine cycles of wet–dry exposure, the UCS of the specimens tended to stabilize. When the specimens were subjected to 15 wet–dry cycles, the UCS decreased from the initial value of 0.96 MPa to 0.36 MPa, representing a total degradation of 62.24%. However, even after 15 cycles, the UCS of the MICP-treated soil remained above the minimum requirement of 0.35 MPa specified in the Chinese standards. The majority of the UCS degradation, 93.87% of the total, occurred within the first nine wet–dry cycles, where the UCS decreased from 0.96 MPa to 0.39 MPa, a reduction of 58.43%. Meanwhile, the UCS of the untreated expansive soil decreased from 0.24 MPa to 0.12 MPa, representing a total degradation of 50%. Although this degradation rate was notably lower than that of the treated soil, the final strength of untreated soil fell significantly below the standard requirement of 0.35 MPa.

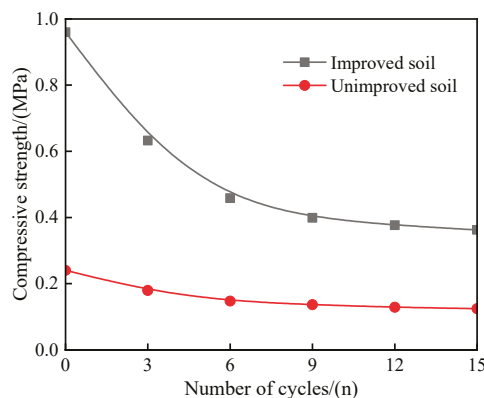


Figure 9. Degradation of UCS results.

The results of the direct shear tests on the specimens after wet–dry cycling are shown in Figure 10. Similarly to the UCS results, the shear strength parameters, namely the cohesion (c) and the angle of internal friction (φ), gradually decreased with an increasing number of wet–dry cycles, also exhibiting a rapid initial decline followed by a more gradual reduction. After 15 wet–dry cycles, the cohesion of the MICP-treated soil samples decreased from 24.34 kPa to 19.74 kPa, a total degradation of 18.92%, while the angle of internal friction decreased from 29.34° to 21.54° , a total degradation of 26.59%. The degradation rate was most pronounced during the first nine wet–dry cycles, with the cohesion decreasing from 24.34 kPa to 20.11 kPa (91.86% of the total degradation) and the angle of internal friction decreasing from 29.34° to 22.45° (88.37% of the total degradation). Despite experiencing greater degradation in cohesion and internal friction angle under wet–dry cycles

than untreated soil, the improved soil maintained its superior shear strength—surpassing untreated soil by 23.9% in cohesion and 71.8% in internal friction angle after 15 cycles.

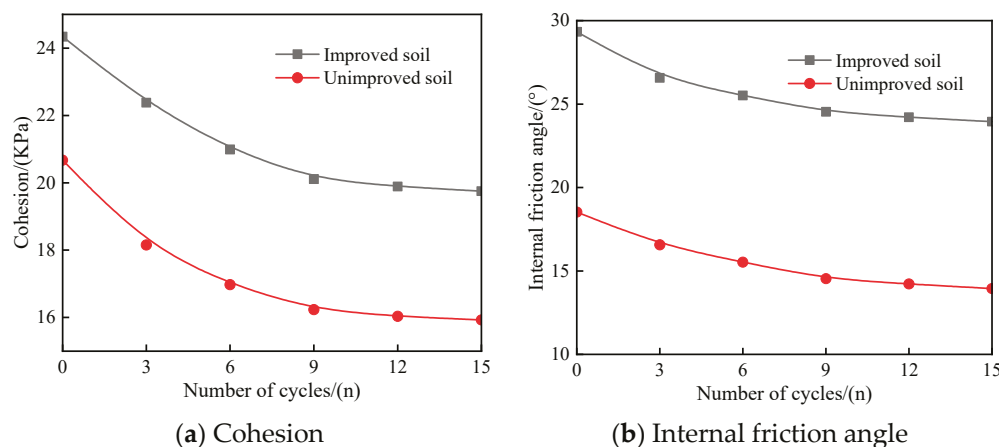


Figure 10. Degradation of shear strength results.

The results of the CBR tests after wet–dry cycling are presented in Figure 11. The CBR value of the MICP-treated soil samples also decreased with an increasing number of wet–dry cycles, following a similar trend of a rapid initial decline and a more gradual reduction. After 15 wet–dry cycles, the CBR value decreased from 8.56% to 5.21%, a total degradation of 39.05%. The majority of the CBR degradation, 91.14% of the total, occurred within the first nine wet–dry cycles, where the CBR value decreased from 8.56% to 5.51%. Meanwhile, the CBR of the untreated expansive soil decreased from 2.05% to 0.77%, representing a total degradation of 62.43%, and the final strength of untreated soil fell significantly below the standard requirement.

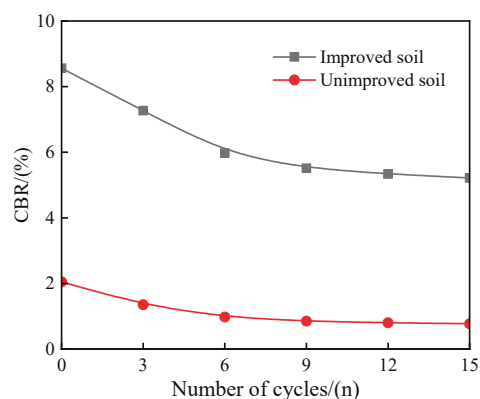


Figure 11. Degradation of CBR results.

These results indicate that the wet–dry cycling had a significant negative impact on the mechanical properties of the MICP-treated expansive soil. The degradation of the mechanical properties, including then UCS, shear strength, and CBR, followed a trend of a rapid initial decline followed by a more gradual reduction. After nine wet–dry cycles, the degradation of the mechanical properties tended to stabilize.

Using untreated expansive soil as a control, the experimental results demonstrate significantly less mechanical property degradation after wet–dry cycles compared to the improved soil. This difference primarily stems from the untreated soil's initially poor mechanical properties, which leave little room for further deterioration. Importantly, even after cyclic wet–dry exposure, the treated soil maintains mechanical properties substantially superior to those of untreated expansive soil.

To further analyze the degradation patterns of the various mechanical parameters, the concept of a damage factor was introduced. The damage factor (D_V) was calculated using Equation (1), based on the initial and post-wet-dry cycling mechanical property values. The total degradation (D_S) and degradation rate (V_S) were then determined using Equations (2) and (3), respectively, as suggested in the literature. The damage factor curves for the different mechanical properties are presented in Figure 12.

$$D_V = 1 - \frac{S}{S_0}, \quad (1)$$

$$D_S = \int_0^a \sqrt{f(x)} dx \quad (2)$$

$$V_S = \frac{\int_0^a \sqrt{1 + [f'(x)]^2} dx}{a} \quad (3)$$

where D_V represents the damage factor; S denotes the mechanical property value after wet-dry cycling; S_0 indicates the initial mechanical property value; D_S signifies total degradation; V_S refers to the degradation rate; a is the number of wet-dry cycles; and $f(x)$ represents the degradation curve for various parameters.

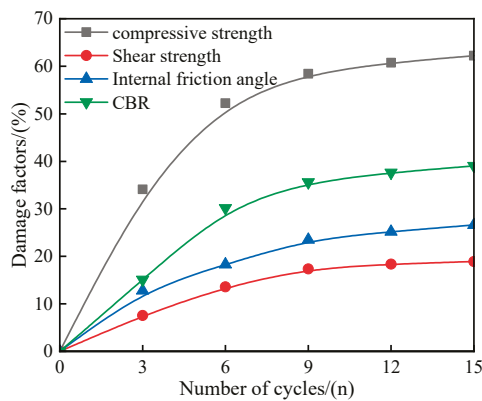


Figure 12. Damage factor curves of mechanical properties.

The total degradation areas for the UCS, angle of internal friction, cohesion, and CBR were calculated to be 710.02, 413.95, 279.24, and 198.89, respectively. The corresponding degradation rates were 3.12, 2.84, 2.49, and 1.97, respectively. These results indicate that the wet-dry cycling had the most significant impact on the unconfined compressive strength of the MICP-treated soil, followed by the CBR, while the shear strength parameters were the least affected.

4.2. Mass Loss

The results of the mass loss tests conducted on the untreated and MICP-treated soil specimens after various wet-dry cycling intervals are presented in Figure 13. The results show that the mass loss rate of the specimens increased with the number of wet-dry cycles, exhibiting a trend of a rapid initial increase followed by a more gradual reduction.

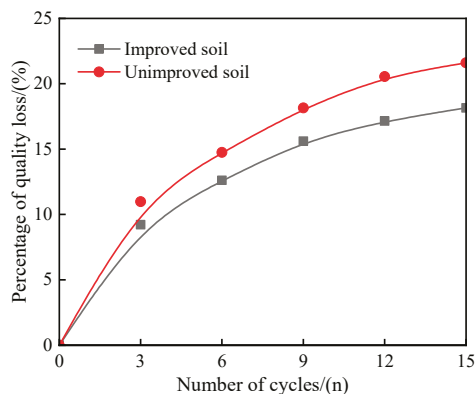


Figure 13. Loss of quality.

After 15 wet–dry cycles, the total mass loss of the MICP-treated soil specimens was 18.15%. However, the majority of the mass loss, 86% of the total, occurred within the first nine wet–dry cycles, where the mass loss rate reached 15.61%. Furthermore, after 15 drying–wetting cycles, the stabilized expansive soil showed an 15.97% lower mass loss rate than the untreated soil.

This behavior can be attributed to the mechanism of mass loss during the wet–dry cycling. When the specimens were submerged in water, the water flow caused the erosion and transportation of some soil particles from the internal structure of the specimens. Additionally, the loose surface particles were also carried away by the water solution, leading to the observed mass loss. After the initial rapid mass loss during the first nine wet–dry cycles, the rate of mass loss gradually decreased. This can be explained by the stabilization of the soil structure, where the remaining particles became more tightly bound together due to the MICP treatment, reducing the susceptibility to further mass loss.

4.3. Volume Expansion

The results of the volume expansion tests conducted on the MICP-treated soil specimens after various wet–dry cycling intervals are presented in Figure 14. The results show that the volume expansion of the specimens increased with the number of wet–dry cycles, exhibiting a rapid initial increase followed by a more gradual reduction.

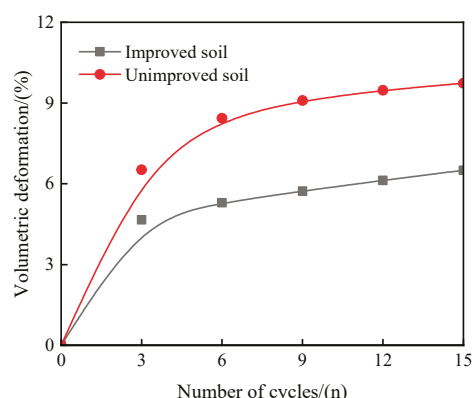


Figure 14. Volume expansion behavior.

After 15 wet–dry cycles, the total volume expansion of the MICP-treated soil specimens was 6.53%. However, the majority of the volume expansion, 71.36% of the total, occurred within the first three wet–dry cycles, where the volume expansion reached 4.66%. Furthermore, after 15 drying–wetting cycles, the stabilized expansive soil exhibited a 33.20% reduction in its volumetric expansion rate compared to the untreated soil.

This behavior can be attributed to the changes in the soil microstructure during the wet–dry cycling. Initially, the MICP treatment had effectively reduced the swelling potential of the expansive soil by altering the internal particle structure and reducing the pore spaces, as observed in the earlier microscopic analysis. However, the subsequent wet–dry cycles caused the progressive degradation of the MICP-induced cementation, leading to an increase in the volume expansion of the soil. After the initial rapid volume expansion during the first three wet–dry cycles, the rate of volume expansion gradually decreased. This can be explained by the stabilization of the soil structure, where the remaining cementation bonds became more resistant to the effects of wet–dry cycling, limiting further volume expansion.

5. Conclusions

This study comprehensively investigated the influence of different cementation solution concentrations on the swelling and mechanical properties of MICP-treated expansive soils. Based on the experimental results, the following main conclusions can be drawn:

1. MICP treatment can significantly improve the swelling characteristics of expansive soils. As the cementation solution concentration increased, the free swell ratio and unconfined swell ratio of the soil samples both exhibited a decreasing trend. When the cementation solution concentration reached 1 M, the swelling properties of the soil samples were substantially suppressed. This can be attributed to the precipitation of calcium carbonate during the MICP process, which fills the pore spaces between soil particles and alters the internal structure, thereby enhancing the anti-swelling performance.
2. MICP treatment also significantly enhanced the mechanical properties of the expansive soil. As the cementation solution concentration increased, the unconfined compressive strength, cohesion, and internal friction angle of the soil samples all exhibited notable improvements. When the cementation solution concentration reached 1 M, the mechanical properties of the soil samples were significantly enhanced, meeting the technical requirements for highway subgrade fill materials. This is primarily due to the filling and cementing effects of the calcium carbonate precipitates, which strengthen the inter-particle bonding within the soil.
3. Under continuous wet–dry cycling, the mechanical properties of the MICP-treated soil samples experienced degradation. The unconfined compressive strength showed the most significant reduction, decreasing by 62.24%. The decreases in cohesion and internal friction angle were relatively smaller, at 18.92% and 26.59%, respectively. The CBR value also decreased by 39.05%. This performance degradation mainly occurred within the first nine wet–dry cycles, after which it tended to stabilize. This is likely due to the partial dissolution and loss of the calcium carbonate precipitates under the repeated wet–dry cycles, leading to a reduction in the strength and stiffness of the MICP-treated soil.

In summary, MICP technology can effectively improve the performance of expansive soils, but in regions with hot and humid climates, the mechanical properties of the treated soil samples may undergo a certain degree of degradation under long-term wet–dry cycling. Therefore, further optimization of the MICP method is needed to enhance its durability under complex environmental conditions, providing a more reliable solution for the engineering construction in expansive soil-prone regions.

Author Contributions: Conceptualization, F.H. and B.J.; methodology, S.W.; software, S.W.; validation, F.H., B.J. and F.Z.; formal analysis, F.Z.; investigation, F.Z.; resources, X.L.; data curation, X.L.; writing—original draft preparation, B.J.; writing—review and editing, B.J. and F.H.; visualization, X.L.; supervision, S.W.; project administration, F.Z.; funding acquisition, S.W. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the Science and Technology Research Project of Chongqing Education Commission of China (Grant numbers: KJQN202305802, KJQN202403413), Chongqing Natural Science Foundation (Grant number: CSTB2024NSCQ-MSX0006).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The raw data supporting the conclusions of this paper will be made available by the authors by request.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Mehmood, M.; Guo, Y.; Liu, Y.; Wang, L.; Nie, W.; Uge, B.U.; Ali, S.; Chen, X.; Zhao, Y. Experimental study on the engineering characteristics of expansive soil improved conjointly using enzyme induced carbonate precipitation and eggshell powder. *Soils Found.* **2025**, *65*, 101567. [CrossRef]
2. Irakoze, D.; Abongo, K.; Waweru, S. Performance of Expansive Soil blended with Waste Marble Dust and Natural Pozzolana for Road Subgrade. *Eng. Technol. Appl. Sci. Res.* **2025**, *15*, 21068–21075. [CrossRef]
3. Luo, P.; Ma, M. Failure Mechanisms and Protection Measures for Expansive Soil Slopes: A Review. *Sustainability* **2024**, *16*, 5127. [CrossRef]
4. Xu, S.; Ma, E.; Lai, J.; Yang, Y.; Liu, H.; Yang, C.; Hu, Q. Diseases failures characteristics and countermeasures of expressway tunnel of water-rich strata: A case study. *Eng. Fail. Anal.* **2022**, *134*, 106056. [CrossRef]
5. Zada, U.; Jamal, A.; Iqbal, M.; Eldin, S.M.; Almoshaogeh, M.; Bekkouche, S.R.; Almuaythir, S. Recent advances in expansive soil stabilization using admixtures: Current challenges and opportunities. *Case Stud. Constr. Mater.* **2023**, *18*, e01985. [CrossRef]
6. Tiwari, N.; Satyam, N.; Puppala, A.J. Strength and durability assessment of expansive soil stabilized with recycled ash and natural fibers. *Transp. Geotech.* **2021**, *29*, 100556. [CrossRef]
7. Al-Gharbawi, A.S.; Najemalden, A.M.; Fattah, M.Y. Expansive soil stabilization with lime, cement, and silica fume. *Appl. Sci.* **2022**, *13*, 436. [CrossRef]
8. Huang, Z.; Sun, H.-Y.; Dai, Y.-M.; Hou, P.-B.; Zhou, W.-Z.; Bian, L.-L. A study on the shear strength and dry-wet cracking behaviour of waste fibre-reinforced expansive soil. *Case Stud. Constr. Mater.* **2022**, *16*, e01142. [CrossRef]
9. Blayi, R.A.; Sherwani, A.F.H.; Mahmod, F.H.R.; Ibrahim, H.H. Influence of rock powder on the geotechnical behaviour of expansive soil. *Int. J. Geosynth. Ground Eng.* **2021**, *7*, 14. [CrossRef]
10. Liu, C.; Lu, K.; Wu, Z.; Liu, X.; Garg, A.; Qin, Y.; Mei, G.; Lv, C. Expansive soil improvement using industrial bagasse and low-alkali ecological cement. *Constr. Build. Mater.* **2024**, *423*, 135806. [CrossRef]
11. Zhang, Y.; Zhao, C.; Zhang, Z.; Zhang, Z.; Kou, J.; Jing, Z. Study on Engineering Characteristics and Mechanism of Expansive Soil Improved by Lime-zeolite Powder. *Acad. J. Archit. Geotech. Eng.* **2024**, *6*, 54–61.
12. Jangid, A.K.; Khatti, J.; Grover, K.S. Effect of multicollinearity in assessing the compaction and strength parameters of lime-treated expansive soil using artificial intelligence techniques. *Multiscale Multidiscip. Model. Exp. Des.* **2025**, *8*, 68. [CrossRef]
13. Por, S.; Nishimura, S.; Likitlersuang, S. Deformation characteristics and stress responses of cement-treated expansive clay Under confined one-dimensional swelling. *Appl. Clay Sci.* **2017**, *146*, 316–324. [CrossRef]
14. Mohamed, A.A.M.S.; Yuan, J.; Al-Ajamee, M.; Dong, Y.; Ren, Y.; Hakuzweyezu, T. Improvement of expansive soil characteristics stabilized with sawdust ash, high calcium fly ash and cement. *Case Stud. Constr. Mater.* **2023**, *18*, e01894. [CrossRef]
15. Zhou, S.; Shi, M.; Chen, W.; Zhang, Y.; Wang, W.; Zhang, H.; Li, D. Improved geotechnical behavior of an expansive soil amended with cationic polyacrylamide. *J. Renew. Mater.* **2021**, *9*, 1941–1957. [CrossRef]
16. Ahmad, S.; Ghazi, M.S.A.; Syed, M.; Al-Osta, M.A. Utilization of fly ash with and without secondary additives for stabilizing expansive soils: A review. *Results Eng.* **2024**, *22*, 102079. [CrossRef]
17. Li, J.; Hou, Q.; Zhang, X.; Zhang, X. Microbial-Induced Mineral Carbonation: A Promising Approach for Improving Carbon Sequestration and Performance of Steel Slag for Its Engineering Utilization. *Dev. Built Environ.* **2025**, *21*, 100615. [CrossRef]
18. Liu, Z.; Beng, J.; Wu, Y.; Nie, K.; Dang, Y.; Yao, Y.; Li, J.; Fang, M. Microbial induced calcite precipitation for improving low-cohesive soil: Mechanisms, methods and macroscopic properties. *Low-Carbon Mater. Green Constr.* **2024**, *2*, 30. [CrossRef]

19. Wei, R.; Xiao, J.; Wu, S.; Cai, H.; Wang, Z. Effectiveness of microbially induced calcite precipitation for treating expansive soils. *Adv. Civ. Eng. Mater.* **2021**, *10*, 350–361. [CrossRef]
20. Chittoori, B.; Neupane, S. Evaluating the application of microbial induced calcite precipitation technique to stabilize expansive soils. In Proceedings of the Civil Infrastructures Confronting Severe Weathers and Climate Changes Conference, Hangzhou, China, 23–25 July 2018; pp. 10–19.
21. Osinubi, K.; Eberemu, A.; Gadzama, E.; Ijimdiya, T. Plasticity characteristics of lateritic soil treated with *Sporosarcina pasteurii* in microbial-induced calcite precipitation application. *SN Appl. Sci.* **2019**, *1*, 825. [CrossRef]
22. Chittoori, B.C.; Rahman, T.; Burbank, M. Microbial-facilitated calcium carbonate precipitation as a shallow stabilization alternative for expansive soil treatment. *Geotechnics* **2021**, *1*, 558–572. [CrossRef]
23. Li, J.; Bi, W.; Yao, Y.; Liu, Z. State-of-the-Art review of utilization of microbial-induced calcite precipitation for improving moisture-dependent properties of unsaturated soils. *Appl. Sci.* **2023**, *13*, 2502. [CrossRef]
24. Huang, W.; Mou, Y.; Li, Y.; Zhao, B.; Li, J.; Wu, X.; Zhou, M. Study on durability of MICP treated cohesive soils Under dry–wet cycle and freeze–thaw cycle. *Arab. J. Geosci.* **2022**, *15*, 422. [CrossRef]
25. Liu, S.; Wen, K.; Armwood, C.; Bu, C.; Li, C.; Amini, F.; Li, L. Enhancement of MICP-treated sandy soils against environmental deterioration. *J. Mater. Civ. Eng.* **2019**, *31*, 04019294. [CrossRef]
26. Tang, C.S.; Liu, B.; Vahedifard, F.; Jiang, N.J.; Zhu, C.; Shen, Z.T.; Pan, X.H.; Cheng, Q.; Shi, B. Enhancing soil resilience to climatic wetting–drying cycles through a bio-mediated approach. *J. Geophys. Res. Earth Surf.* **2024**, *129*, e2023JF007573. [CrossRef]
27. ASTM D698-12; Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft³ (600 kN-m/m³)). ASTM International: West Conshohocken, PA, USA, 2021.
28. ASTM D4318-17e1; Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. ASTM International: West Conshohocken, PA, USA, 2018.
29. ASTM D4546-21; Standard Test Methods for One-Dimensional Swell or Collapse of Soils. ASTM International: West Conshohocken, PA, USA, 2021.
30. ASTM D1883-21; Standard Test Method for California Bearing Ratio (CBR) of Laboratory-Compacted Soils. ASTM International: West Conshohocken, PA, USA, 2021.
31. ASTM D2166; Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. ASTM International: West Conshohocken, PA, USA, 2024.
32. ASTM D3080; Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions. ASTM International: West Conshohocken, PA, USA, 2023.
33. Li, X.; Zhang, C.; Xiao, H.; Jiang, W.; Qian, J.; Li, Z. Reducing Compressibility of the Expansive Soil by Microbiological-Induced Calcium Carbonate Precipitation. *Adv. Civ. Eng.* **2021**, *2021*, 8818771. [CrossRef]
34. Tian, X.; Xiao, H.; Li, Z.; Li, Z.; Su, H.; Ouyang, Q. Experimental study on the strength characteristics of expansive soils improved by the MICP method. *Geofluids* **2022**, *2022*, 3089820. [CrossRef]
35. Tang, C.-S.; Yin, L.-Y.; Jiang, N.-J.; Zhu, C.; Zeng, H.; Li, H.; Shi, B. Factors affecting the performance of microbial-induced carbonate precipitation (MICP) treated soil: A review. *Environ. Earth Sci.* **2020**, *79*, 94. [CrossRef]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Engineering Performance and Mechanism of Alkali-Activated Ground Granulated Blast Furnace Slag–Zeolite Powder Grouting Materials

Longni Wang ^{1,2}, Hongyuan Fu ^{1,2}, Qianfeng Gao ^{1,2,*}, Jintao Luo ³, Jing Tang ³, Jianping Song ⁴, Youjun Li ⁴ and Guangtao Yu ^{1,2}

¹ School of Transportation, Changsha University of Science & Technology, Changsha 410114, China; wangln116@163.com (L.W.)

² National Key Laboratory of Green and Long-Life Road Engineering in Extreme Environment, Changsha 410114, China

³ School of Civil and Environmental Engineering, Changsha University of Science & Technology, Changsha 410114, China

⁴ Guangxi Transportation Science and Technology Group Co., Ltd., Nanning 530007, China

* Correspondence: qianfeng.gao@csust.edu.cn

Abstract: Geopolymer-based grouting materials often have a higher early strength, better durability, and lower environmental impact than those of traditional cement-based grouts. However, existing geopolymer grouts face common challenges such as rapid setting and low compatibility with treated substrates. This study develops a new grouting material using industrial byproducts to overcome these limitations while optimizing performance for reinforcing silty mudstone slopes. The base materials used were ground granulated blast furnace slag (GGBFS) and zeolite powder, with calcium lignosulphonate (CL) serving as the retarding agent and NaOH as the alkali activator. The investigation focused on the effects of the mix ratio and water–binder ratio on the setting time, flowability, bleeding rate, concretion rate, and compressive strength of the new grouting material. Scanning electron microscope (SEM) and X-ray diffraction (XRD) analyses were employed to examine the action mechanism of the material components in the slurry. The one-factor standard deviation method and Grey Relational Analysis (GRA) were used to assess the influence of each material component on the slurry performance indices and the correlation between each performance index and its optimal mix ratio. Subsequently, the optimal mix ratio of the new grouting material was ascertained. The results indicate that the setting time is positively correlated with the zeolite powder and CL dosages and the water–binder ratio, while it is inversely related to the NaOH dosage. The flowability is significantly enhanced with increasing zeolite powder and NaOH dosages, but decreases at a higher CL dosage and water–binder ratio. This insight is crucial for optimizing the workability of the grouting material under various conditions. The optimal ratio of the grout is zeolite powder:GGBFS:CL:NaOH = 30:70:5:7, with a water–binder ratio of 0.6. Compared to existing commercial grouting materials, the compressive strength of this new grout is comparable to that of silty mudstone. This significantly reduces the problem of stress concentration at the grout–rock interface due to strength differences, thus effectively reducing the risk of secondary cracking at the interface. These findings provide a new material solution for grouting and repairing fractured silty mudstone slopes.

Keywords: alkali activation; GGBFS–zeolite powder; grouting materials; mechanical properties; mechanisms

1. Introduction

The continuous development of transportation infrastructure in China has led to a substantial increase in the number of silty mudstone slopes along roads and railways in the hot and humid southern regions. However, frequent rainfall and evapotranspiration can significantly contribute to the deterioration of silty mudstone slopes, potentially leading to slope failures [1]. This poses a significant threat to the safety of road operations and the well-being of individuals and property. Grouting techniques are commonly used to repair fractured silty mudstone slopes, and their effectiveness depends largely on the type of grouting material used. Presently, silicate cement-based grouting material remains the predominant material employed for slope grouting due to its inherent advantages, including stability and high strength. However, its production incurs substantial costs and results in substantial carbon dioxide emissions during the manufacturing process, a factor that is incongruent with China's commitment to achieving its strategic 'dual-carbon' target. Therefore, there is an urgent need to develop green, low-carbon, cost-effective grouting materials that are efficient for restoration and environmentally friendly.

Geopolymer is a gelling material with a three-dimensional mesh structure, consisting of AlO_4 and SiO_4 tetrahedral structural units. It is made from minerals or industrial wastes rich in silica and aluminum through the chemical reaction process of depolymerization, reorganization, coalescence, and setting in an alkaline environment [2,3]. In recent years, a significant number of studies have been conducted on the use of geopolymer grouting materials as a replacement for silicate cement-based grouting materials. Yingli Khan et al. [4] investigated the feasibility of using waste glass powder (GP) as a partial precursor in alkali-activated mortars made from fly ash (FA) and ground granulated blast furnace slag (GGBFS). The findings demonstrate that the FA-GGBFS mortar exhibits a dense and compact microstructure at a 20% GP dosing. Mukhtar et al. [5] used mechanochemical activation to prepare geopolymer grouting material from GGBFS and fly ash, and found that the flowability and mechanical properties of the slurry significantly improved with an increase in the slag dosage. In the aforementioned studies on the preparation of geopolymer grouting materials, the majority of the main cementitious materials selected include fly ash, GGBFS, and other highly reactive solid waste materials. However, fly ash is associated with several disadvantages, including a protracted setting time and inadequate durability. Additionally, it contains high levels of radioactive substances and heavy metals, which can easily pollute soil and groundwater if not properly managed, posing challenges to addressing the root of environmental issues [6–8]. This limits its use in engineering applications [9]. Although GGBFS has high activity, rich resource reserves, and a low cost [10,11], its dry shrinkage is significant, and it is difficult to meet engineering requirements when using it alone [12]. Zeolite powder is selected as a composite material with GGBFS powder for the preparation of geopolymer grout slurry. Zeolite powder is a material produced by grinding natural zeolites. It is of interest because of its local availability and low cost [13]. The unique spatial mesh structure of this material provides a large internal surface area and reactivity [14], which can effectively improve the performance and quality of the grouting material [15], and, at the same time, overcome the difficulties of low raw material activity and poor alkali activation in building materials from solid wastes.

As demonstrated in previous studies, geopolymer grouting materials are most commonly deployed in road construction, filling hollow areas, and reinforcement tunnels. Rahman et al. [16] conducted a study on fly ash (FA) and ground granulated blast furnace slag (GGBFS)-based self-compacting geopolymer concrete (SCGC), and assessed its efficacy in marine applications. The findings indicated that the mixture can be effectively used in marine environments, with a design strength of 40 MPa attained under harsh marine conditions. Ni et al. [17] developed a new type of synchronous grouting material for tunnel

grouting and determined the optimal mix ratio of geopolymer grouting material through a series of performance tests and Response Surface Methodology (RSM). This material was subsequently applied to metro projects. In summary, geopolymer grouting materials have demonstrated excellent performance and application potential. However, existing research on grouting materials does not fully meet the reinforcement needs for silty mudstone slopes. The strength disparity of existing geopolymer- and cement-based grouting materials and silty mudstone is too significant. This renders the process of grouting repair susceptible to secondary cracking, which has the potential to compromise the efficacy of the repair and the long-term stability of the slope [18]. As a result, there is a research gap in the development and application of geopolymer grouting materials for silty mudstone slope reinforcement.

To address this gap, this study develops a new environmentally friendly grouting material using GGBFS and zeolite powder as the base materials, CL as the retarder, and NaOH solution as the alkali activator. The tests focused on the setting time, flowability, bleeding rate, concretion rate, and compressive strength and used scanning electron microscopy (SEM) and X-ray diffraction (XRD). The one-factor standard deviation and Grey Relational Analysis (GRA) were used to quantitatively analyze the influence of different material components and performance indicators on the mix ratio, and determine the optimal mix ratio of alkali-activated GGBFS–zeolite powder grouting materials. These findings could provide new materials for the grouting reinforcement of fractured rocky slopes in the southern region of China.

2. Material Properties

2.1. Base Materials

GGBFS and zeolite powder were selected as geopolymer base materials due to their environmental friendliness, optimized properties, and wide range of applications [19,20]. Compared with the elevated energy consumption of metakaolin, the restricted availability of silica fume, and the concomitant environmental hazards linked with fly ash, GGBFS possesses the benefit of water-hardening and performance-enhancing properties as a byproduct of iron and steel. The zeolite powder stands out for its porosity, adsorption, and energy-saving features that do not require calcination. Therefore, industrial solid waste (S95 GGBFS powder and 200 mesh zeolite powder) was used as the base material for the preparation of the grouting material, and the materials are shown in Figure 1.

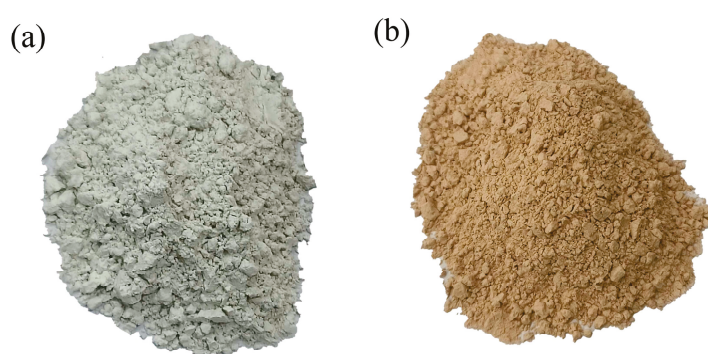


Figure 1. Basic materials: (a) GGBFS powder; (b) zeolite powder.

S95 GGBFS powder, with a specific surface area of $412 \text{ m}^2/\text{kg}$, a density of $2.9 \text{ g}/\text{cm}^3$, a specific gravity of 2.45, and a particle size range of $5\text{--}20 \text{ }\mu\text{m}$, was purchased from Hengyuan New Material Company, located in Zhengzhou City, Henan Province, China. The 200-mesh natural zeolite powder was purchased from Henan Gongyi Hengxin Co., Ltd., China (located in Gongyi City, Henan Province, China), with a density range of $1.75 \text{ g}/\text{cm}^3$, a

specific surface area of 560 m²/kg, a specific gravity of 2.1, and a particle size range of 74–100 µm. Figure 2 shows the XRD patterns of the primary base materials, while Table 1 provides a comprehensive overview of their chemical compositions. As shown in Figure 2 and Table 1, the primary chemical composition of GGBFS powder consists of SO₂ and CaO, accounting for 72.35%, followed by Al₂O₃, at 15.04%. The predominant minerals include calcite (CaCO₃), anhydrite (CaSO₄), and calcium alumina feldspar (Ca₂Al₂SiO₇). The primary chemical composition of zeolite powder consists of SO₂, accounting for 73.35% of the total, followed by Al₂O₃, at 12.45%. The predominant mineral compositions are gismondine (CaAl₂Si₂O₈·4H₂O) and wairakite (CaAl₂Si₆O₁₆·6H₂O).

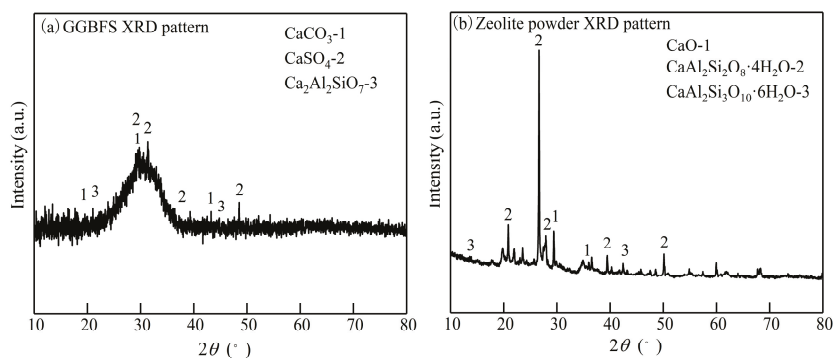


Figure 2. XRD of base materials: (a) GGBFS powder; (b) zeolite powder.

Table 1. Chemical composition of base materials.

Materials	SO ₂ /%	CaO/%	Al ₂ O ₃ /%	MgO/%	SO ₃ /%	Other/%
GGBFS powder	32.96	39.39	14.04	10.96	1.9	0.75
Zeolite powder	73.35	4.23	12.45	0.84	-	9.13

2.2. Additives

White NaOH granules were used to prepare the solution as an alkali activator, aimed at stimulating the activity of GGBFS and zeolite powder and promoting the coagulation and hardening of the slurry. NaOH broke the Si-O and Al-O bonds by providing OH⁻ ions, breaking them and producing ions with activity (e.g., silicate ions and aluminate ions) which, under certain conditions, polymerised to form C-(A)-S-H or N-A-S-H gels. This would promote the coagulation and hardening of the slurry [21].

The setting time of the GGBFS–zeolite powder grouting material was generally rapid, failing to meet the criteria in the ‘Common Portland cement’ (GB 175-2023) standard [22]. To address this, CL powder was used as a retarder, extending the hydration reaction time and increasing the setting time of the slurry. Calcium lignosulphonate is a multi-component polymer anionic surfactant with the molecular formula C₂₀H₂₄CaO₁₀S₂. It is a dark brown powder with an aromatic odor and is easily soluble in water. It is also stable, with strong dispersibility, adhesion, and chelating properties.

3. Test Schemes

A reasonable setting time ensures sufficient working time during construction while allowing the slurry to set and cure quickly after grouting. Good flowability enables the slurry to uniformly diffuse and effectively fill the complex cracks in the rock slope. A low bleeding rate maintains the slurry’s volume stability and homogeneity during solidification, preventing internal pores and strength loss from water dissipation. When the compressive strength of the slurry closely matches that of the silty mudstone, stress concentration is minimized, reducing the risk of secondary cracking. Therefore, grouting materials with

appropriate compressive strength, good flowability, low bleeding rate, and reasonable setting time are crucial for effective slope reinforcement and long-term stability [23,24].

To determine the optimal mix ratio, preliminary tests on setting time and compressive strength were conducted on specimens with various mix ratios. The results of these preliminary tests indicated the following ranges for further optimization: 0% to 50% zeolite powder, 50% to 100% GGBFS, 1% to 5% CL, 1% to 9% NaOH, and 0.4 to 0.8 of the water–binder ratios. In the formal experiment, the influence of the zeolite powder-to-GGBFS ratio on grout performance was initially evaluated at 2% CL, 7% NaOH, and a 0.6 water–binder ratio. Subsequently, the dosages of CL and NaOH, as well as the water–binder ratio, were varied, respectively, while maintaining the optimal zeolite powder-to-GGBFS ratio. A total of 18 mix ratio schemes were designed for the formal experiment, as shown in Table 2.

Table 2. Alkali-activated GGBFS–zeolite powder grouting material ratio.

Groups	Zeolite Powder Dosage/%	GGBFS Powder Dosage/%	CL Dosage/%	NaOH Dosage/%	Water–Binder Ratio
1	0	100			
2	10	90			
3	20	80			
4	30	70	2	7	0.6
5	40	60			
6	50	50			
7	Optimal zeolite powder-to-GGBFS ratio		1		
8			3		
9			4	7	0.6
10			5		
11				1	
12			3	3	0.6
13				5	
14				9	
15					0.4
16			3	7	0.5
17					0.7
18					0.8

Although there is no well-developed and unified standard system for testing alkali-activated materials, as there is for conventional cementitious materials, the workability testing principles for alkali-activated materials are similar to those for conventional materials. Therefore, the setting time tests, flow tests, and bleeding rate tests of alkali-activated GGBFS–zeolite powder grouting materials were conducted in strict accordance with the following test methods: the Test Methods for Water Requirement of Normal Consistency, Setting Time and Soundness of Portland Cement (GB/T 1346-2011) [25]; the Test Method for Fluidity of Cement Mortar (GB/T 2419-2005) [26]; and the Testing Methods of Cement and Concrete for Highway Engineering (JTG 3420-2020) [27]. To reduce the potential impact of test errors, a total of three parallel specimens were designed for each group, and the mean values were subsequently calculated after the removal of outliers. The compressive strength of the slurry stone body was evaluated in accordance with the Test Method of Cement Mortar Strength (GB/T 17671-2021) [28]. Demolded specimens were then subjected to a curing process for 7, 14, and 28 days under standard conditions (temperature of 20 ± 2 °C and relative humidity >95%). Subsequently, the mechanical properties were determined using a WDW-100C microcomputer-controlled electronic universal testing machine. This tester was produced by Shanghai Hualong Testing Instrument Co. China (Shanghai, China).

The specimen size was 40 mm × 40 mm × 40 mm, using a three-test mold casting block. Following the compressive strength test, the specimen fragments were subjected to grinding, and the powder was subsequently analyzed using XRD. Fragments with dimensions less than 12 mm were selected for the SEM test, which was performed using a Zeiss EVO10 instrument to observe the surface micro-morphology of the stone bodies. The tester was produced by Carl Zeiss and is located in Jena, Germany.

4. Results and Discussion

4.1. Setting Time

As demonstrated in Figure 3, the setting time of the slurry was influenced by various factors. The present study investigated the effect of different dosages of zeolite powder on the setting time of the slurry. The findings indicated a direct correlation between the increased dosage of zeolite powder and the rise in setting time. The initial setting time increased from 20 min to 36 min, an 80% increase, while the final setting time rose from 62 min to 109 min, indicating an increase of 75.81%. The phenomenon was due to the porous nature of zeolite powder. This characteristic facilitated the absorption of water molecules, which were then introduced into the internal pores. Consequently, the hydration reaction rate of the slurry was inhibited, resulting in a prolonged setting time.

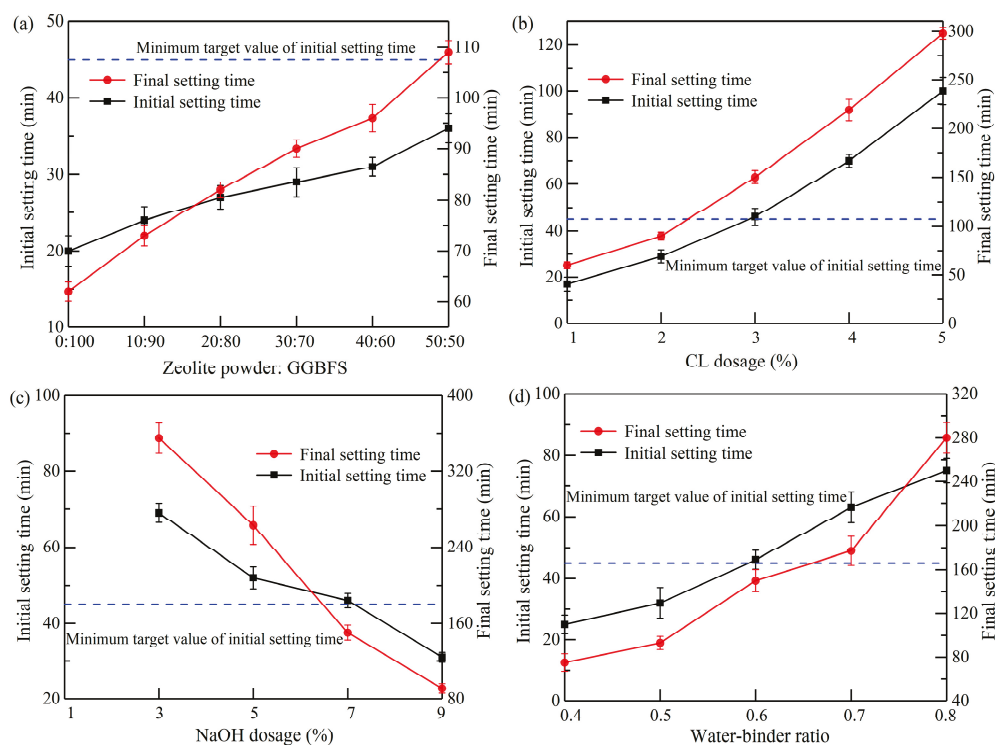


Figure 3. Effects of different factors on setting time: (a) zeolite powder: GGBFS; (b) CL dosage; (c) NaOH dosage; (d) water-binder ratio.

Figure 3b shows that when the CL dosage was set at 5%, the initial and final setting times of the slurry were maximized to 100 and 298 min, respectively, with an increased rate of 488.24% and 396.67%. This is because the CL molecule contains a large number of hydrophilic groups (e.g., sulphonic acid groups and hydroxyl groups), which promote the interaction between the CL molecules and the surface of solid particles [29,30]. When mixed with slurry containing solid particles, such as zeolite powder and GGBFS, the CL molecules were adsorbed onto the surface of the solid particles by chemisorption. During this process, a complexation reaction occurred between the CL molecules and Ca^{2+} , forming

a gelling film. This film hindered the direct contact of zeolite powder and GGBFS with water molecules. Consequently, the hydration reaction was slowed down.

It was established that as the quantity of NaOH increased, the setting time correspondingly decreased. The initial and final setting times decreased by 55.07% and 74.37%, respectively. Notably, the slurry remained unsolidified at a NaOH dosage of 1%. This outcome demonstrates that the alkaline environment was not sufficient to stimulate the production of a gelatinous substance in the slurry when the NaOH dosage was set at 1%, as per the mix ratio established in Group 11. An increased water–binder ratio resulted in an increased setting time for the slurry. Furthermore, when the water–binder ratio was between 0.7 and 0.8, the rate of increase in the slurry setting time was accelerated, with a rise of 19.05% and 58.19%, respectively. The hydration reaction of the slurry was delayed as the free water molecules in the slurry increased, leading to an increased setting time.

The final results showed that, among the 18 groups of ratios, the initial and final setting times of the slurries in Groups 1–7, 11, and 14–16 did not meet the requirements of the Chinese technical specification standard GB 175-2023 (initial setting time was not less than 45 min and final setting time was not more than 600 min).

4.2. Flowability

The flowability was identified as the primary metric for evaluating the performance of the grouting material, with the hydration reaction rate serving as a key factor influencing flowability. Figure 4 demonstrates the impact of various factors on slurry flowability. It was established that, as the dosage of zeolite powder increased, the flowability of the slurry gradually decreased, reaching a minimum value of 28.43 cm. The increase in the specific surface area of the compound grouting material, resulting from the substantial replacement of GGBFS by zeolite powder, led to a decrease in the average water content per unit surface area. This, in turn, led to excessively dry or viscous characteristics in the prepared slurry, thereby reducing its flowability. Moreover, an increase in slurry flowability was observed with a 17.86% rise in the CL dosage. In the field of construction materials, CL is used as a water-reducing agent. Under constant water–binder ratio conditions, the dosage of CL is increased, resulting in elevated levels of free water molecules within the slurry.

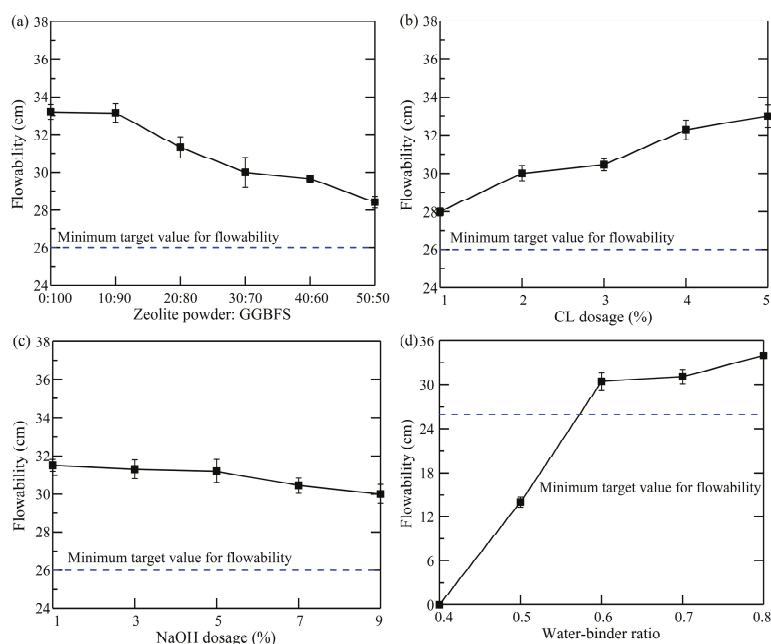


Figure 4. Effects of different factors on flowability: (a) zeolite powder–GGBFS; (b) CL dosage; (c) NaOH dosage; (d) water–binder ratio.

As the NaOH dosage increased, the slurry flowability decreased more gradually, from 31.5 cm to 30 cm. Accordingly, the impact of the NaOH dosage on flowability was found to be negligible, and the resulting flowability values met the established minimum standard criteria. When the water–binder ratio was 0.6, the flowability of the slurry showed a substantial enhancement, exhibiting a 117.5% increase in comparison to the water–binder ratio of 0.5. The test pattern was similar to that in the results reported by Gasperi et al. [31]. The greater number of water molecules in the slurry led to their attraction to each other, forming a smooth interface. As a result, this enhanced the flowability of the slurry. According to the Chinese technical specification GB/T 50448-2015 [32], the flowability of the slurry should be not less than 260 mm. Therefore, Groups 15 and 16 did not meet this standard.

4.3. Bleeding and Concretion Rate

The bleeding rate directly influenced the stability and filling effect of the slurry, while the concretion rate directly affected the strength and durability of the grouting material [33]. Figure 5 provides a visual representation of the effect of different factors on the slurry's bleeding and concretion rates. As the dosage of zeolite powder increased, the bleeding rate of the slurry gradually increased, while the concretion rate steadily decreased. When the ratio of zeolite powder to GGBFS was 50:50, the bleeding rate of the slurry peaked, with a reduction of 100%. The slurry bleeding rate increased uniformly with elevated CL dosage. Due to the retardation and water-reducing properties of CL, the slurry hydration reaction was retarded, leading to an increase in free water molecules and the bleeding rate. The slurry bleeding rate gradually decreased as the NaOH dosage increased. The increased alkali concentration in the solution promoted a variety of chemical reactions in the slurry [34]. Concurrently, a significant amount of heat was released, thereby facilitating the progression of the reaction. Consequently, the bleeding rate was minimal, while the concretion rate was maximal at a NaOH dosing of 9%.

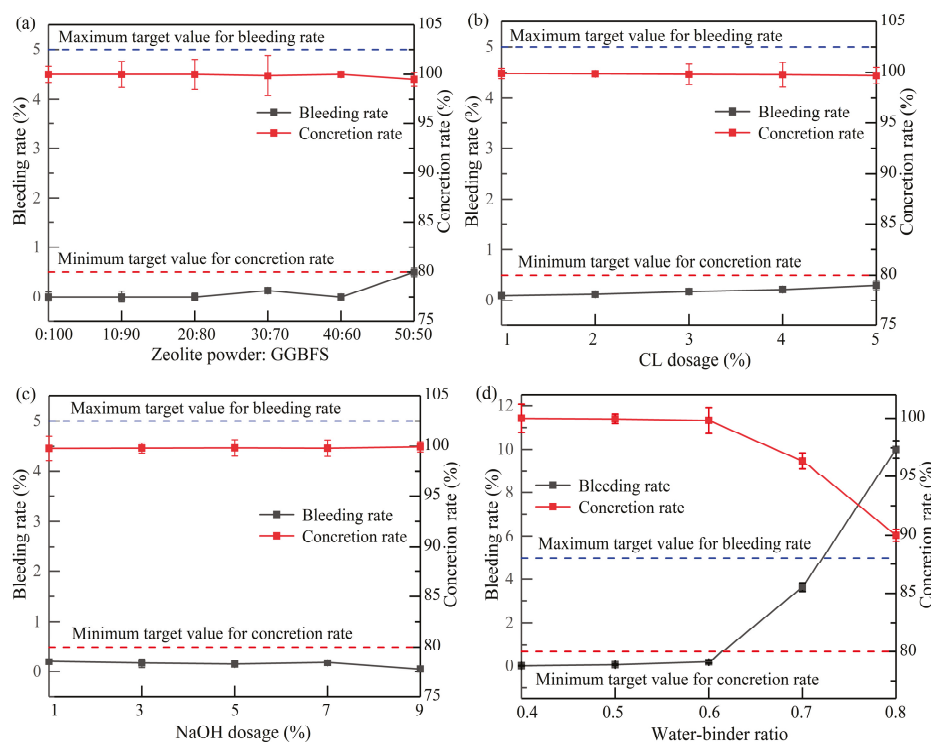


Figure 5. Effects of different factors on bleeding and concretion rate: (a) zeolite powder–GGBFS; (b) CL dosage; (c) NaOH dosage; (d) water–binder ratio.

As shown in Figure 5d, an increase in the water–binder ratio led to a gradual increase in the slurry’s bleeding rate. As the water–binder ratio increased from 0.7, a significant increase in the bleeding rate was observed, reaching its maximum at a water–binder ratio of 0.8. Due to the increased number of free water molecules in the slurry that did not participate in the chemical reaction, the setting time increased, particles precipitated faster, and the bleeding rate increased. The final results showed that only the bleeding rate of Group 18 exceeded the upper limit value specified in the Chinese technical specification DL/T 5148-2021 [35].

4.4. Compressive Strength of Stone Bodies

The compressive strength is a pivotal metric for assessing the material properties of alkali-activated GGBFS–zeolite powder slurry. As shown in Figure 6, the compressive strength of the slurry is influenced by various factors. When the ratio of zeolite powder to GGBFS was 50:50, the 28 d compressive strength of the stone bodies reached a minimum value of 12.49 MPa. The incorporation of zeolite powder significantly reduced the total pore volume of the slurry. Furthermore, the internal structure and composition of the pores were optimized, and the stone bodies were densified. However, due to the zeolite powder’s high water absorption and storage capacity [36], a consequent high level of water absorption occurred during the chemical reaction, which hindered the hydration reaction. It was observed that when CL was dosed at 5%, the 28 d compressive strength of the slurry stone bodies reached a maximum of 17.19 MPa, 26.30% higher than the lowest value. CL was hypothesized to exert an air-entraining effect [37], which was theorized to promote the hydration reaction, increase the product, decrease porosity, and improve the compressive strength of the slurry stone bodies.

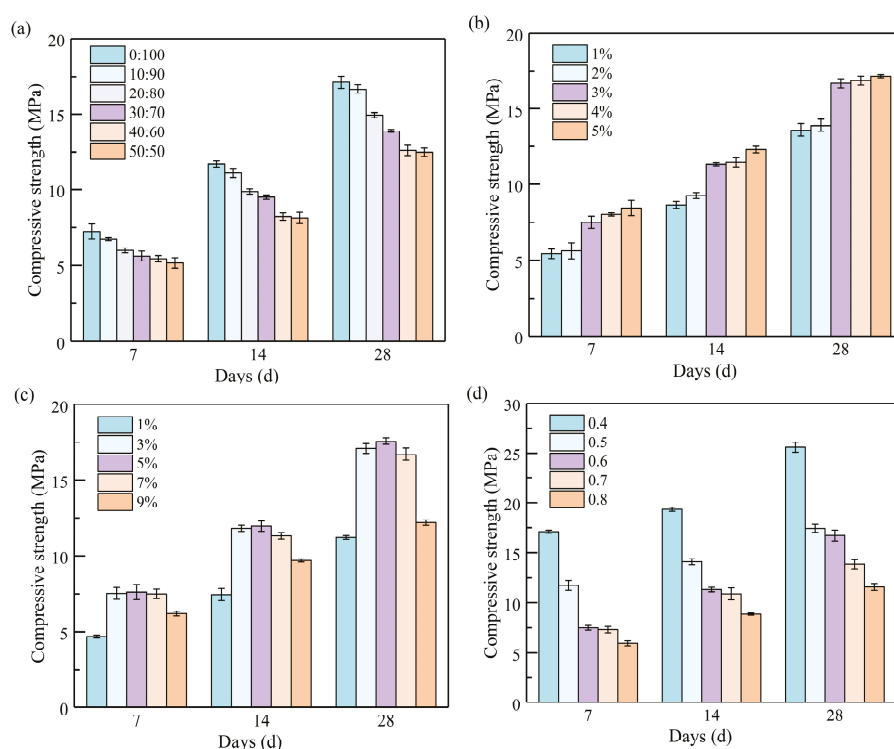


Figure 6. Effects of different factors on the compressive strength of grouting slurry stone: (a) zeolite powder–GGBFS; (b) CL dosage; (c) NaOH dosage; (d) water–binder ratio.

As the NaOH dosage, increased, the compressive strength of the slurry stone bodies first increased, then decreased. A maximum value of the 28 d compressive strength of 17.59 MPa was attained at a dosage of 5%, and the maximum increase in the 28 d

compressive strength was 52.49% at a dosage of 3%. Thus, the correct amount of alkali activator promoted the binding of active molecules in the chemical reaction [38]. As the NaOH dosage exceeded 5%, the elevated OH^- concentration resulted in a rapid reaction rate, leading to the generation of numerous hydration products in a short period. These products accumulated rapidly on the surface of the active particles, impeding the ensuing reaction and consequently diminishing the strength of the stone bodies. This observation is consistent with the results reported by Mahfoud et al. [39] and Huang et al. [40].

The compressive strength of the stone bodies decreased with the water–binder ratio. The 28 d compressive strength decreased most significantly at a water–binder ratio of 0.5, reaching 31.80%. As the water–binder ratio increased, the free water molecules in the slurry also increased. In addition, the excess free water molecules diluted the alkaline material in the system and reduced the OH^- concentration. This reduction slowed the rate of the hydration reaction, limiting the formation of hydration products and their ability to adequately fill the pores, further reducing the compressive strength of the stone bodies [41,42].

4.5. XRD and SEM Analysis

As shown in Figure 7, the XRD analysis was conducted on samples with varying zeolite powder-to-slag powder ratios (0:100, 30:70, and 50:50), CL retarder dosages (2% and 5%), and NaOH alkali activator dosages (3% and 9%). The main products of the alkali-activated GGBFS–zeolite powder grouting materials were $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$, C-S-H, SiO_2 , and CaCO_3 . As C-(A)-S-H is an amorphous phase [43,44], its diffraction peaks were not evident in the XRD analysis.

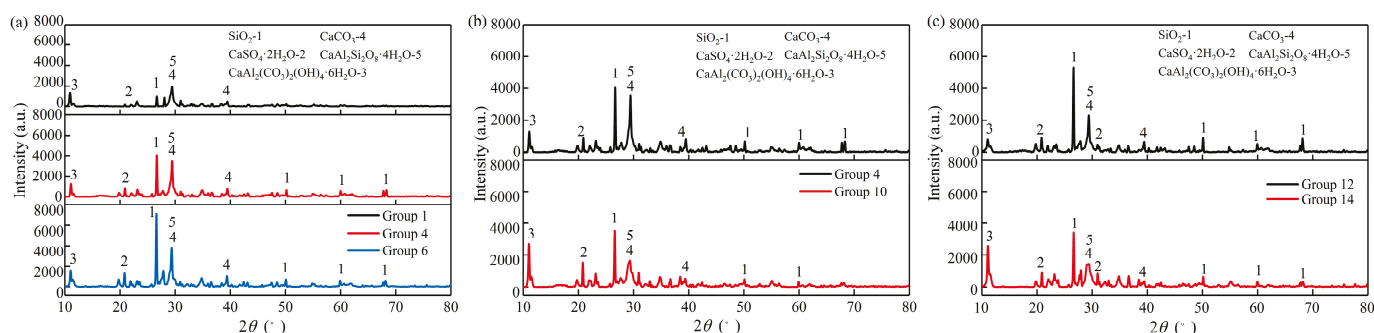


Figure 7. Effects of different factors on XRD: (a) zeolite powder–GGBFS; (b) CL dosage; (c) NaOH dosage.

As the dosage of zeolite powder increased from 0% to 30%, both the CaCO_3 and SiO_2 diffraction peaks increased, with a rise of 79.51% and 344.29%, respectively. When the dosage of zeolite powder reached 50%, the SiO_2 diffraction peaks continued to enhance, reaching a level of 76.92%, while the CaCO_3 diffraction peaks remained largely unchanged, increasing by only 9.46%. The findings indicated that in an alkaline environment, where all other factors remained constant, increasing the dosage of zeolite powder resulted in an increased content of unconsumed SiO_2 . This, in turn, led to a decreased pH value of the slurry, thereby hindering the effective hydration of the gelling material. In this case, the high-energy-state aluminum–oxygen bond ($-\text{Al}-\text{O}-$) and silicon–oxygen bond ($-\text{Si}-\text{O}-$) exhibited difficulty in dissolution [45], resulting in a decline in the generated C-S-H and C-(A)-S-H cementitious hydrides. This, in turn, contributed to the gradual decrease in the compressive strength of the slurry stone bodies with increased dosage of zeolite powder.

It was demonstrated that as the dosage of CL increased, the intensities of the SiO_2 and CaCO_3 diffraction peaks decreased significantly, with a degree of reduction of 12.74% and 52.70%, indicating that SiO_2 and CaCO_3 were gradually converted into other hydration products. CL, an anionic surfactant, enhances the water dispersibility of adsorbent

materials [46]. This improvement was attributed to the ability of CL to wet the surface of the particles, thereby facilitating the hydration reaction. Furthermore, the diffraction peak intensities of the monocalcium aluminate hydration products exhibited a concomitant enhancement with increasing CL doping, showing an increase of 114.90%. This suggests that the monocalcium aluminate hydration products play a substantial role in enhancing the densification of stone bodies [47]. This was the same as the conclusion reached in Section 4.4. Keeping all other conditions constant, the strength of the stone bodies doped with 5% CL exceeded that of those doped with 2%.

The SEM images of the slurry stone bodies in Figure 8 combined with Figure 6b clearly show that the compressive strength of the stone bodies was higher at a 5% CL dosage compared to 3%. The specimen with 5% CL doping had a more compact structure, fewer holes, denser gels, and stronger bonding of the gels to the skeleton. In addition to the Calcium Sulfoaluminate hydrate (C-(A)-S-H) gelation phase and Cement Silicate Hydrogel (C-S-H) gelation phase inside the stone bodies [48], the mineral backbone in the stone bodies consisted of a number of unreacted minerals such as quartz and calcite.

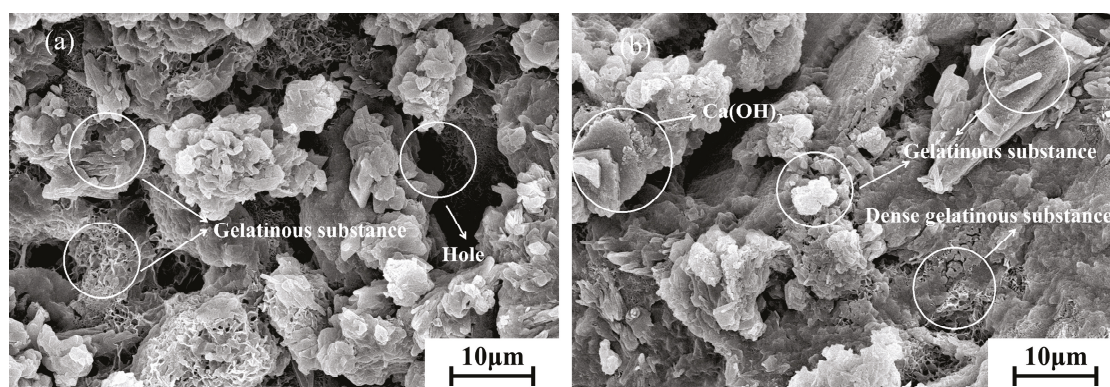


Figure 8. SEM image of alkali-activated GGBFS–zeolite powder grouting slurry stone: (a) 2% CL; (b) 5% CL.

In the presence of the NaOH alkali activator, the SiO_2 and Al_2O_3 crystal structures dissolved to form aluminum–silicon complexes, which were rearranged to form a stable three-dimensional mesh structure. On the other hand, gypsum (CaSO_4), the main mineral in GGBFS, reacted with NaOH and ambient CO_2 to form CaCO_3 . As depicted in Figure 7c, when the NaOH dosage increased, the XRD peak area of CaCO_3 rose by 20.51%, accompanied by an increase in its crystal content. Moreover, the intensity of the diffraction peaks of the monocalcium aluminate hydration products also witnessed a significant growth of 226.19%. When the alkali content was too high, OH^- would be consumed during the hydration of the geopolymer and there would be an excess of Na^+ ions. This makes it difficult to maintain charge equilibrium, preventing the polymerization reaction from progressing and leading to the formation of a less robust stone body. This phenomenon explained why the overall intensity remained unaltered, despite the decreased intensity of the SiO_2 diffraction peaks by 35.38%.

5. Evaluation of Material Factors and Performance Indicators by One-Way Standard Deviation and GRA

The objective of this section is to ascertain the impact of each material factor on the slurry performance indicators and the correlation between each performance indicator of the slurry and its optimal slurry ratio. The ultimate objective was to determine the optimal mix ratio of the grout. The key to determining the optimal mix ratio of alkali-activated GGBFS–zeolite powder grouting material was quantifying the effect of four

material factors on performance indicators. The four performance indicators were the setting time, flowability, bleeding rate, and compressive strength. The four material factors were the ratio of the zeolite powder to the GGBFS, the set retarder dosage, the alkali activator dosage, and the water–binder ratio. The one-way standard deviation method was used to assess the influence of each material factor on the performance indicators. Subsequently, Grey Relational Analysis (GRA) was used to evaluate the correlation between each performance indicator and the optimal mix ratio, aiming to determine the optimal mix ratio.

5.1. One-Way Standard Deviation Analysis

Performance data from single-factor tests at varying levels were used to calculate the standard deviations of the four performance indicators: setting time, flowability, bleeding rate, and compressive strength. Specifically, among the 18 ratio groups, Groups 1–6 were used for evaluating the effect of the zeolite powder–GGBFS ratio; Groups 4 and 7–10 were used for the CL dosage; Groups 8 and 11–14 were used for the NaOH dosage; and Groups 8 and 15–18 were used for the water–binder ratio. Standard deviation was used to assess the influence of different material factors on the performance indicators. The results are presented in Figure 9. The magnitude of the standard deviation provides an intuitive representation of the degree of impact, in contrast to the less intuitive nature of judging from the variance value [49,50]. When considering a single factor, it simplifies the analysis by avoiding the complexities of multi-factor analysis, allowing the research to remain focused.

$$S = \sqrt{\frac{1}{n} \sum_{i=1}^n (X_i - \bar{X})^2} \quad (1)$$

where X_i and $\bar{X}$ are the sample values and sample mean values of the performance indicators obtained by testing the single factor at different levels, respectively; n is the number of different factor levels.

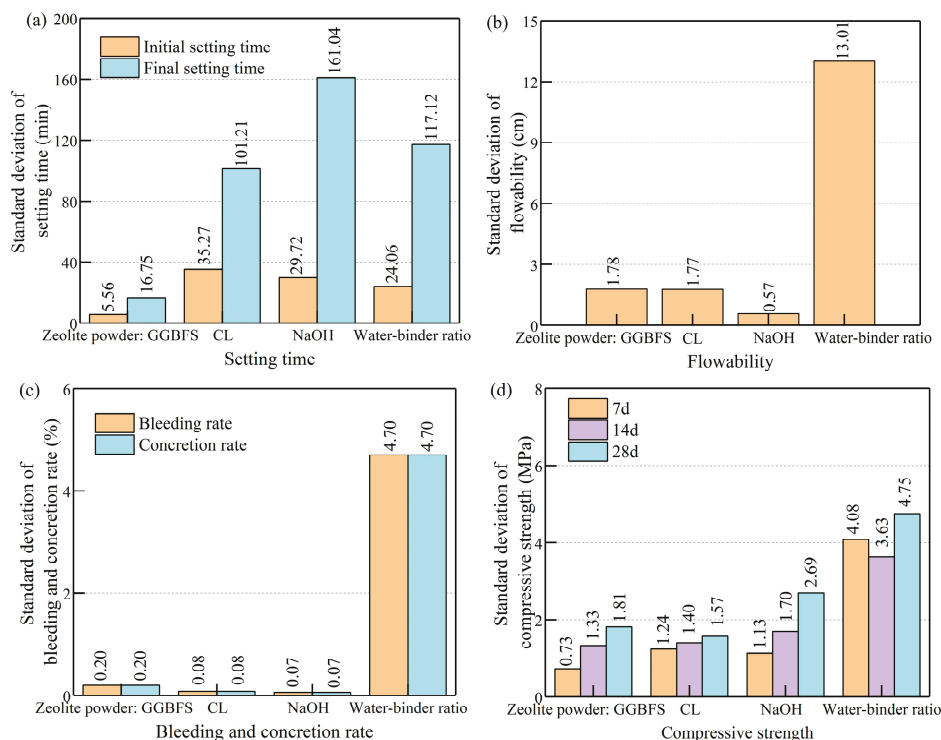


Figure 9. Degree of influence of different factors on performance: (a) setting time; (b) flowability; (c) bleeding rate and concretion rate; (d) compressive strength.

As shown in Figure 9, the dosage of CL was the primary factor influencing the initial setting time of the slurry. The dosage of NaOH was a key factor in determining the final setting time. The water–binder ratio was the most significant parameter affecting flowability. The bleeding rate and concretion rate were primarily influenced by the water–binder ratio, followed by the dosage ratio of CL and zeolite powder to GGBFS. Regarding the compressive strength at 7, 14, and 28 days, the water–binder ratio was the most important factor, followed by the dosage of NaOH.

5.2. GRA-Based Evaluation of Impact Factors

Grey Relational Analysis (GRA) was used to assess the relevance of each performance indicator on the grouting material and to determine the optimal mix ratio [51,52]. It was found that, according to the relevant specifications [22,32,35] and field requirements [53,54] for the setting time, flowability, bleeding rate, and compressive strength, among the 18 ratio groups, the initial setting times of Groups 1 to 7, 11, and 14 did not meet the minimum standards. Groups 15 and 16 did not meet the minimum requirements for the initial setting time and flowability. The bleeding rate of Group 18 exceeded the upper limit, and the compressive strength of Group 17 did not meet the specified minimum standards. Five groups satisfied the specified criteria, and the GRA evaluated only these five groups. The following details were provided:

According to the stipulated specifications, the performance indicators (initial setting time, final setting time, flowability, bleeding rate, and compressive strength) of the selected ratio group (1–18) had to be met. This was achieved by constructing a comparative series $D_i(j)$, where $j = 1, 2, \dots, m$ (m is the number of groups meeting the specification) and $I = 1, 2, \dots, n$ (n is the number of control performance indicators).

To ensure the accuracy of the results and avoid the effects of significant differences between the data, the data were preprocessed by normalization, as shown in Equation (2).

$$d_i(j) = \frac{D_i(j) - \min_i}{\max_i - \min_i} \quad (2)$$

According to the standard [22,32,52] and site requirements, the optimal range for the performance indicators is as follows: initial setting time of 45–120 min, final setting time of 150–600 min, flowability of 26–33 cm, bleeding rate of 0–5%, and compressive strength of 12–18 MPa.

The construction of the referenced series $Z_0(j)$ is shown in Equation (3).

$$Z_0(j) = \max(d_{1j}, d_{2j}, \dots, d_{nj}) \quad (3)$$

The grey relational coefficient ξ_i is shown in Equation (4).

$$\xi_i[Z_0(j), d_i(j)] = \frac{\min_i \min_j |Z_0(j) - d_i(j)| + \rho \max_i \max_j |Z_0(j) - d_i(j)|}{|Z_0(j) - d_i(j)| + \rho \max_i \max_j |Z_0(j) - d_i(j)|} \quad (4)$$

where ρ is the grey discrimination coefficient, taken as 0.5. In a large number of practical applications and studies, it is found that setting the discrimination coefficient to 0.5 often provides a more reasonable reflection of the true correlation between factors in most cases [55,56].

The degree of serial correlation between the series $D_i(j)$ and the reference series Z_i was compared using the correlation r_i , as shown in Equation (5). The correlation degree of each performance index of the grouting material was then determined.

$$r_i = \frac{1}{n} \sum_{j=1}^n \xi_i[Z_0(j), d_i(j)] \quad (5)$$

where the closer the correlation r_i is to 1, the better the series $D_i(j)$ is correlated to the performance metrics.

The indicator weights W_i , as shown in Equation (6), should be used. The optimal mix ratio group was then determined, as shown in Table 3.

$$W_i = \frac{r_i}{\sum_{j=1}^n r_j} \quad (i = 1, 2, \dots, n) \quad (6)$$

Table 3. Normalized weighted total scores for grouting materials.

Groups	Initial Setting Time	Final Setting Time	Flowability	Bleeding Rate	Compressive Strength	Weighted Total Score	Normalized Weighted Score
8	0.045	0.000	0.788	0.882	0.925	0.533	0.107
9	1.116	0.645	1.112	1.078	0.960	0.982	0.196
10	2.455	1.383	1.240	1.471	1.017	1.507	0.301
12	1.071	1.916	0.939	0.833	1.001	1.150	0.230
13	0.313	1.056	0.921	0.735	1.097	0.828	0.166

As shown in Figure 10, the correlation of the alkali-activated GGBFS–zeolite powder slurry was investigated. The correlation of the compressive strength was found to be the highest and closest to 1, indicating that this performance indicator had the greatest correlation with optimum proportioning. Except for the compressive strength correlation, which exceeded 0.7, the correlations of the other four performance indicators were all greater than 0.5, and all were correlated with the optimal mix ratio. The weights of the performance indicators (initial setting time, final setting time, flowability, bleeding rate, and compressive strength) on the mix ratio were 19.71%, 19.72%, 19.98%, 19.67%, and 20.92%, respectively. The correlation of each performance index of the grouting material on the optimal mix ratio was, in descending order, as follows: compressive strength, flowability, final setting time, initial setting time, and bleeding rate.

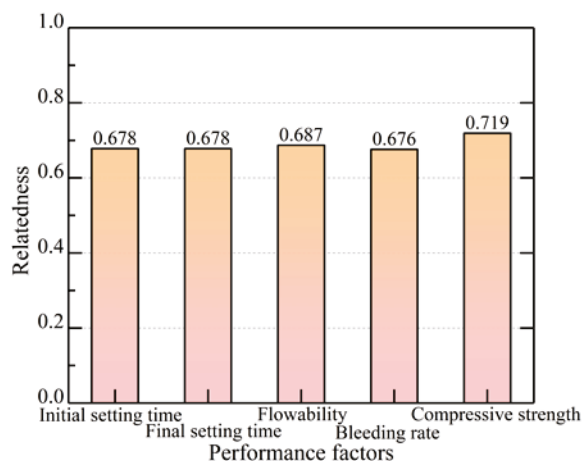


Figure 10. Correlation of different performance indicators.

The priority given to compressive strength over the other factors is due to the primary objective of grouting being the enhancement of the load-bearing capacity of silty mudstone slopes. When the compressive strength is inadequate, the slopes may fail under loading even if the other parameters meet the requirements. The final setting time needs to be reasonably controlled to avoid slurry loss, but a slightly longer period of time is acceptable to meet the construction schedule and to achieve the required strength. Flowability determines whether the slurry can be injected smoothly, but focusing on it alone and ignoring strength is not effective for reinforcement. Similarly, the bleeding rate and initial setting time have an effect, but to a lesser extent than the strength when the basic properties and construction operations are ensured. Considering the relative importance of these factors, the weight of each performance indicator in determining the grouting effectiveness was comprehensively assessed. The optimum ratio was determined using the indicator weights from Group 10, with a zeolite powder:GGBFS:CL:NaOH ratio of 30:70:5:7 and a water–binder ratio of 0.6.

6. Conclusions

In this study, the engineering performance regarding the setting time, flowability, bleeding rate, concretion rate, and compressive strength of alkali-activated GGBFS–zeolite powder slurry with different mix ratios were measured. The surface morphology and chemical composition of slurry concretions was analyzed using SEM and XRD. Simultaneously, the influence of various material factors and performance indicators on the slurry was quantitatively analyzed using one-factor standard deviation and GRA. The main conclusions are as follows:

- (1) The setting time of alkali-activated GGBFS–zeolite powder slurry exhibits a positive correlation with the proportions of zeolite powder and CL and the water–binder ratio, while it is inversely related to the amount of NaOH. Notably, when the NaOH dosage is 1%, the slurry fails to set. With an increase in the dosage of zeolite powder and NaOH, the fluidity of the slurry is significantly enhanced. Conversely, a higher CL dosage and water–binder ratio lead to a decline in flowability and an increase in the slurry's viscosity. This insight is crucial for maximizing the workability of the grouting material under varying conditions.
- (2) The compressive strength of the slurry stone bodies gradually decreases with an increasing zeolite powder dosage and water–binder ratio. Conversely, an increase in the CL dosage enhances the compressive strength of the stone bodies. At the same time, the impact of NaOH doping on the compressive strength of the stone bodies initially increases, followed by a subsequent decrease. At a 5% doping level, the compressive strength of the stone bodies reaches a maximum of 17.59 MPa after 28 days. The compressive strength of this grout is compatible with silty mudstone, making it suitable for reinforcing silty mudstone slopes without the risk of secondary cracking along the material interface.
- (3) In the case of alkali-activated GGBFS–zeolite powder slurry, the water–binder ratio is identified as a key factor influencing performance indices. The correlation between each performance index of the grouting material and the optimal mix ratio is, in descending order, as follows: compressive strength, flowability, final setting time, initial setting time and bleeding rate. The optimal mix ratio is zeolite powder:GGBFS:calcium lignosulphonate (CL):NaOH = 30:70:5:7, with a water–binder ratio of 0.6. This mix ratio not only meets the specification requirements but also provides a cost-effective and environmentally sustainable solution, utilizing widely available industrial byproducts.

Future efforts will focus on validating the optimal alkali-activated GGBFS–zeolite powder slurry ratio for practical use, alongside studies on shrinkage, durability, and adaptability to different construction environments. Additionally, life cycle assessments or CO₂ emission comparisons with silicate cement will be conducted to evaluate the environmental sustainability of the slurry.

Author Contributions: Conceptualization, L.W., H.F. and Q.G.; data curation, G.Y. and J.L.; formal analysis, L.W. and J.L.; funding acquisition, H.F. and L.W.; investigation, J.S. and Y.L.; methodology, Q.G. and J.T.; project administration, Q.G.; resources, J.T.; validation, L.W. and G.Y.; visualization, G.Y.; writing—original draft, L.W. and J.L.; writing—review and editing, H.F., Q.G. and Y.L. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the National Natural Science Foundation of China (Nos. 52378440, 42477143); the Natural Science Foundation of Hunan Province (No. 2023JJ10045); the Key Science and Technology Program in the Transportation Industry (Nos. 2022-MS1-032, 2022-MS5-125); the Project Fund Practical Innovation and Entrepreneurship Enhancement Program of Changsha University of Science and Technology, China (No. CLSJCX23009); and the Guangxi Key Research and Development Program (No. AB23075184).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The original contributions presented in the study are included in the article, further inquiries can be directed to the corresponding author.

Acknowledgments: The authors gratefully acknowledge the financial support of Wenguang Wang and Ling Zeng for this study. And special thanks also go to Key Laboratory of Road Structure and Material Ministry of Communication.

Conflicts of Interest: Authors Jianping Song and Youjun Li were employed by the company Guangxi Transportation Science and Technology Group Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

Abbreviations

The following abbreviations are used in this manuscript:

GGBFS	Ground granulated blast furnace slag
CL	Calcium lignosulphonate
GP	Glass powder
FA	Fly ash
SEM	Scanning electron microscope
XRD	X-ray diffraction
GRA	Grey Relational Analysis
C-S-H	Cement Silicate Hydrogel
C-(A)-S-H	Calcium Sulfoaluminate Hydrates

References

1. Fu, H.Y.; Jiang, H.B.; Qiu, X.; Ji, Y.P.; Chen, W.; Zeng, L. Seepage characteristics of a fractured silty mudstone under different confining pressures and temperatures. *J. Cent. South Univ.* **2020**, *27*, 1907–1916. [CrossRef]
2. Davidovits, J. Geopolymers and geopolymeric materials. *J. Therm. Anal.* **1989**, *35*, 429–441. [CrossRef]
3. Matsimbe, J.; Dinka, M.; Olukanni, D.; Musonda, I. Geopolymer: A Systematic Review of Methodologies. *Materials* **2022**, *15*, 6852. [CrossRef]
4. Khan, M.N.N.; Kuri, J.C.; Sarker, P.K. Effect of waste glass powder as a partial precursor in ambient cured alkali activated fly ash and fly ash-GGBFS mortars. *J. Build. Eng.* **2021**, *34*, 101934. [CrossRef]

5. Abed, M.H.; Abbas, I.S.; Hamed, M.; Canakci, H. Rheological, fresh, and mechanical properties of mechanochemically activated geopolymer grout: A comparative study with conventionally activated geopolymer grout. *Constr. Build. Mater.* **2022**, *322*, 126338. [CrossRef]
6. Zhang, K.; Yang, Y.; Li, H. Cementitious properties of coal-based metakaolin prepared from coal gangue via Fe₃O₄-Assisted microwave activation. *J. Clean. Prod.* **2024**, *448*, 141277. [CrossRef]
7. Hamada, H.M.; Abed, F.; Katman, H.Y.B.; Humada, A.M.; Al Jawahery, M.S.; Majdi, A.; Yousif, S.T.; Thomas, B.S. Effect of silica fume on the properties of sustainable cement concrete. *J. Mater. Res. Technol.* **2023**, *24*, 8887–8908. [CrossRef]
8. Pan, S.; Li, J.; Gong, H.; Zhu, Z.; Xu, S.; Jiang, C.; Cai, W. Resource Disposal and Products of Fly Ash from Domestic Waste Incineration in Zhejiang Province, China: Migration and Change of Hazardous Heavy Metals. *Sustainability* **2024**, *16*, 302. [CrossRef]
9. Teixeira, E.R.; Camoes, A.; Branco, F.G.; Aguiar, J.B.; Figueiro, R. Recycling of biomass and coal fly ash as cement replacement material and its effect on hydration and carbonation of concrete. *Waste Manag.* **2019**, *94*, 39–48. [CrossRef]
10. Loke, C.K.; Lehane, B.; Aslani, F.; Majhi, S.; Mukherjee, A. Non-Destructive Evaluation of Mortar with Ground Granulated Blast Furnace Slag Blended Cement Using Ultrasonic Pulse Velocity. *Materials* **2022**, *15*, 6957. [CrossRef]
11. Sitarz, M.; Hager, I.; Choińska, M. Evolution of Mechanical Properties with Time of Fly-Ash-Based Geopolymer Mortars under the Effect of Granulated Ground Blast Furnace Slag Addition. *Energies* **2020**, *13*, 1135. [CrossRef]
12. Xiang, J.C.; He, Y.; Liu, L.P.; Zheng, H.; Cui, X.M. Exothermic behavior and drying shrinkage of alkali-activated slag concrete by low temperature-preparation method. *Constr. Build. Mater.* **2020**, *262*, 120056. [CrossRef]
13. Kouchachvili, L.; Bardy, D.A.; Djebbar, R.; Hogg, L.E. Natural zeolites as host matrices for the development of low-cost and stable thermochemical energy storage materials. *J. Porous Mater.* **2023**, *30*, 163–173. [CrossRef]
14. Hu, H.B.; He, Z.H.; Fan, K.J.; Shibro, T.G.; Liu, B.J.; Shi, J.Y. Properties enhancement of recycled coarse aggregates by pre-coating/pre-soaking with zeolite powder/calcium hydroxide. *Constr. Build. Mater.* **2021**, *286*, 122888. [CrossRef]
15. Mijailović, N.R.; Nedić Vasiljević, B.; Ranković, M.; Milanović, V.; Uskoković-Marković, S. Environmental and Pharmacokinetic Aspects of Zeolite/Pharmaceuticals Systems—Two Facets of Adsorption Ability. *Catalysts* **2022**, *12*, 837. [CrossRef]
16. Rahman, S.K.; Al-Ameri, R. Marine Geopolymer Concrete-A Hybrid Curable Self-Compacting Sustainable Concrete for Marine Applications. *Appl. Sci.* **2022**, *12*, 3116. [CrossRef]
17. Ni, Z.L.; Wang, S.Y.; Zheng, X.C.; Qi, C.H. Application of geopolymer in synchronous grouting for reusing of the shield muck in silty clay layer. *Constr. Build. Mater.* **2024**, *419*, 135345. [CrossRef]
18. Le, H.; Sun, S.; Kulatilake, P.H.; Wei, J. Effect of grout on mechanical properties and cracking behavior of rock-like specimens containing a single flaw under unconfined compressive. *Int. J. Geomech.* **2018**, *18*, 04018129. [CrossRef]
19. Li, Q.T.; Li, Z.G.; Yuan, G.L. Effects of elevated temperatures on properties of concrete containing ground granulated blast furnace slag as cementitious material. *Constr. Build. Mater.* **2012**, *35*, 687–692. [CrossRef]
20. Feng, B.; Zhang, Y.; Li, W.; Che, G.; Liu, C.; Jv, A.; Lu, J.; Sun, W.; Guan, R. Efficient photocatalytic removal of ciprofloxacin through in-situ growth of g-C₃N₄ on the surface of clinoptilolite. *J. Phys. Chem. Solids* **2024**, *192*, 112077. [CrossRef]
21. Qu, J.S.; Li, H.Q.; Li, S.P.; Hou, X.J.; Chang, R.Q.; Zhang, Y.F. Coal gasification slag-derived highly reactive silica for high modulus sodium silicate synthesis: Process and mechanism. *Chem. Eng. J.* **2024**, *479*, 1385–8947. [CrossRef]
22. GB 175-2023; Common Portland Cement; State Administration for Market Regulation; National Standardization Administration. Standards Press of China: Beijing, China, 2023.
23. Yu, H.; Yi, Y.L.; Unluer, C. Heat of hydration, bleeding, viscosity, setting of Ca(OH)₂-GGBS and MgO-GGBS grouts. *Constr. Build. Mater.* **2021**, *270*, 121839. [CrossRef]
24. Zhang, J.; Wang, H.; Awed, A.M.; Xie, J.; Gu, F. Performance evaluation of high early strength micro-expansion geopolymer grout potentially used for sustainable road infrastructure. *Transp. Geotech.* **2024**, *49*, 101400. [CrossRef]
25. GB/T 1346-2011; Test Methods for Water Requirement of Normal Consistency, Setting Time and Soundness of the Portland Cement; General Administration of Quality Supervision; Inspection and Quarantine of the People's Republic of China; Standardization Administration of the People's Republic of China. Standards Press of China: Beijing, China, 2011.
26. GB/T 2419-2005; Test Method for Fluidity of Cement Mortar; General Administration of Quality Supervision; Inspection and Quarantine of the People's Republic of China; Standardization Administration of the People's Republic of China. Standards Press of China: Beijing, China, 2005.
27. JTG 3420-2020; Testing Methods of Cement and Concrete for Highway Engineering; Ministry of Transport of the People's Republic of China. China Communication Press: Beijing, China, 2020.
28. GB/T 17671-2021; Test Method of Cement Mortar Strength; State Administration for Market Regulation; Standardization Administration of the People's Republic of China. Standards Press of China: Beijing, China, 2021.
29. Jiang, J.; Wang, F.; Wang, L.; Zhang, J.; Wang, L. Experimental investigation on effects of polyaniline-modified-lignin on cement-based materials: Strength, hydration and dispersion. *Constr. Build. Mater.* **2023**, *394*, 131853. [CrossRef]

30. Magina, S.; Barros-Timmons, A.; Evtuguin, D.V. Synthesis of Lignosulfonate-Based Dispersants for Application in Concrete Formulations. *Materials* **2021**, *14*, 7388. [CrossRef]
31. de Gasperi, J.; Dörr, G.; Melchior, E.F.; Longhi, M.A.; de Matos, P.R.; Rodríguez, E.D. Effect of activator type and concentration, water-to-solid ratio, and time on the flowability of metakaolin-based geopolymer pastes. *J. Mater. Civ. Eng.* **2022**, *34*, 04022205. [CrossRef]
32. GB/T 50448-2015; Technical Code for Application of Cementitious Grout; Ministry of Housing and Urban-Rural Development of the People's Republic of China; General Administration of Quality Supervision, Inspection and Quarantine of the People's Republic of China. China Architecture & Building Press: Beijing, China, 2015.
33. Topçu, I.B.; Atesin, Ö. Effect of high dosage lignosulphonate and naphthalene sulphonate based plasticizer usage on micro concrete properties. *Constr. Build. Mater.* **2016**, *120*, 189–197. [CrossRef]
34. Liu, C.H.; Li, Z.Y.; Bezuijen, A.; Chen, L.H.; Cachim, P. Optimizing the shield tunnel backfilling grouts with supplementary cementitious materials by response surface methodology. *Constr. Build. Mater.* **2024**, *421*, 135575. [CrossRef]
35. DL/T 5148-2021; Technical Specification for Cement Grouting of Hydraulic Structures; Ministry of Water Resources of the People's Republic of China. China Water & Power Press: Beijing, China, 2021.
36. Kantarci, F.; Maras, M.M. Fabrication of Novel Geopolymer Grout as Repairing Material for Application in Damaged RC Beams. *Int. J. Civ. Eng.* **2022**, *20*, 461–474. [CrossRef]
37. Touloumet, Q.; Postole, G.; Massin, L.; Lorentz, C.; Auroux, A. Investigation of the impact of zeolite shaping and salt deposition on the characteristics and performance of composite thermochemical heat storage systems. *J. Mater. Chem. A* **2023**, *11*, 2737–2753. [CrossRef]
38. Liang, G.W.; Yao, W.; She, A.M. Rheology and microstructure of lithium slag/metakaolin geopolymer pastes: Insights from particle packing and water dynamic evolution. *J. Build. Eng.* **2024**, *95*, 110261. [CrossRef]
39. Mahfoud, E.; Ndiaye, K.; Maherzi, W.; Aggoun, S.; Abriak, N.E.; Benzerzour, M. Thermodynamic modeling of one-part-geopolymer binders based on fly ash and micronized dredged sediments. *Constr. Build. Mater.* **2024**, *452*, 138906. [CrossRef]
40. Huang, D.; Chen, H.; Zou, Y.; Yuan, Q.; Peng, H. Influence of raw material properties on microscopic and mechanical characteristics of alkali-activated materials. *Case Stud. Constr. Mater.* **2024**, *20*, e03319. [CrossRef]
41. Aliabdo, A.A.; Elmoaty, A.A.E.M.; Salem, H.A. Effect of water addition, plasticizer and alkaline solution constitution on fly ash based geopolymer concrete performance. *Constr. Build. Mater.* **2016**, *121*, 694–703. [CrossRef]
42. Wan, Q.; Zhang, Y.M.; Zhang, R.B. The effect of pore behavior and gel structure on the mechanical property at different initial water content. *Constr. Build. Mater.* **2021**, *309*, 125146. [CrossRef]
43. Bauchy, M.; Qomi, M.J.A.; Ulm, F.J.; Pellenq, R.J.M. Order and disorder in calcium-silicate-hydrate. *J. Chem. Phys.* **2014**, *140*, 214503. [CrossRef] [PubMed]
44. Ruengsillapanun, K.; Udtaranakron, T.; Pulngern, T.; Tangchirapat, W.; Jaturapitakkul, C. Mechanical properties, shrinkage, and heat evolution of alkali activated fly ash concrete. *Constr. Build. Mater.* **2021**, *299*, 123954. [CrossRef]
45. Zhou, C.S.; Tang, Q.; Lu, L. Al Content Effect on Microstructure and Strength in Calcium Aluminosilicate Hydrate Chain Integration. *Strength Mater.* **2022**, *54*, 929–941. [CrossRef]
46. Li, J.Q.; Zhang, W.X.; Garbev, K.; Monteiro, P.J.M. Coordination environment of Si in calcium silicate hydrates, silicate minerals, and blast furnace slags: A XANES database. *Cem. Concr. Res.* **2021**, *143*, 106376. [CrossRef]
47. Liu, M.F.; Zhu, J.L.; Zhang, C.Y.; He, P.; Chen, D.X.; Zhong, G.J.; Liu, Q.; Hu, W.Y.; Chen, Y.Z.; Zhu, J.Y. Effect of calcium lignosulfonate on surface modification and bioleaching of chalcocopyrite. *Biochem. Eng. J.* **2024**, *207*, 109329. [CrossRef]
48. Rambo, D.A.S.; Ukrainczyk, N.; Silva, F.D.; Koenders, E.; Toledo, R.D.; Gomes, O.D.M. Calcium-aluminate mortars at high temperatures: Overcoming adverse conversion effects using clinker aggregates. *Cem. Concr. Compos.* **2019**, *96*, 212–224. [CrossRef]
49. Abdrakhmanov, V.I.; Dobrotin, S.A.; Kosyreva, O.N.; Logutov, V.I. Statistical Evaluation of the Standard Deviation of Chromatographic Retention Time in the Capillary Column Temperature Programming Mode. *J. Anal. Chem.* **2021**, *76*, 641–652. [CrossRef]
50. Wu, P.; Zhang, Q.; Wang, G.; Yang, F.; Xue, F. Dynamic feature selection combining standard deviation and interaction information. *Int. J. Mach. Learn. Cybern.* **2023**, *14*, 1407–1426. [CrossRef]
51. Zhang, S.; Qiao, W.; Wu, Y.; Fan, Z.; Zhang, L. Multi-Response Optimization of Ultrafine Cement-Based Slurry Using the Taguchi-Grey Relational Analysis Method. *Materials* **2021**, *14*, 117. [CrossRef] [PubMed]
52. Abid, N.; Ikram, M.; Wu, J.Z.; Ferasso, M. Towards environmental sustainability: Exploring the nexus among ISO 14001, governance indicators and green economy in Pakistan. *Sustain. Prod. Consum.* **2021**, *27*, 653–666. [CrossRef]
53. Yu, G.; Fu, H.; Gao, Q.; Zeng, L.; Chen, J.; Ma, C. Development and Application of Flue Gas Desulfurized Gypsum and Blast-Furnace-Slag-Based Grouting Material for Cracked Silty Mudstone. *Materials* **2024**, *17*, 5975. [CrossRef] [PubMed]
54. Yu, X.W.; Fu, H.Y.; Zeng, L.; Liu, J.; Qiu, X.Y. Damage constitutive model for soft rocks and its experimental verification on silty mudstone considering cyclic rock–water interactions. *Bull. Eng. Geol. Environ.* **2024**, *83*, 254. [CrossRef]

55. Liao, Y.; Zheng, W.; Long, J.; Xie, X.; Hu, J. Optimization of Backrest Skeleton of Carbon Fiber Reinforced Plastic Car Seat Based on Grey Euclidean Relational Analysis Method. *Int. J. Automot. Technol.* **2023**, *24*, 1189–1203. [CrossRef]
56. Li, Y.; Zhao, S.; Chang, J.; Hou, H.; Chen, C.; Cheng, F.; He, N.; Wu, X. Using Taguchi grey relational analysis to optimize the dimensional parameters of a Coercivity detection probe. *Sāadhanā* **2023**, *48*, 249. [CrossRef]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Optimization of Exposed Aggregate Concrete Mix Proportions for High Skid Resistance and Noise Reduction Performance

Xudong Zha ^{1,*}, Chengzhi Wu ¹, Runzhou Luo ¹ and Yaqiang Liu ²

¹ School of Traffic and Transportation Engineering, Changsha University of Science and Technology, No. 2-960 Wanjiali Rd. (South), Yuhua District, Changsha 410114, China

² Department of Railway Engineering, School of Civil Engineering, Central South University, 68 South Shaoshan Road, Changsha 410075, China

* Correspondence: zhaxd410@163.com

Abstract: Conventional cement concrete pavements often suffer from rapid skid resistance degradation and excessive traffic noise, necessitating effective solutions. This study investigates exposed aggregate concrete (EAC) through orthogonal experimental methods to evaluate the effects of four mix design parameters—water–binder ratio, sand ratio, coarse aggregate volume ratio, and proportion of aggregates > 9.5 mm—on surface texture characteristics, skid resistance and noise reduction (SRNR) performance, and mechanical properties. The optimal EAC mix proportions were developed, and the correlations between surface texture characteristics and SRNR performance were established. Results indicate that the proportion of aggregates > 9.5 mm significantly influences surface texture characteristics and SRNR performance. The optimal mix proportions (water–binder ratio: 0.43, sand ratio: 31%, coarse aggregate volume ratio: 42%, and proportion of aggregates > 9.5 mm: 50%) exhibited superior mechanical properties, achieving a 31.5% increase in pendulum value and a 6.48 dB reduction in tire/surface noise compared to grooved conventional concrete. The noise reduction frequency range is mainly concentrated in the mid-high frequency range of 1.5~4.0 kHz, which is more sensitive to the human ear. High correlations were observed between the surface texture characteristics and SRNR performance. Specifically, noise value decreased progressively with increasing exposed aggregate depth, while the pendulum value exhibited a trend of initial decrease, followed by an increase and subsequent decrease in response to the elevated exposed aggregate area ratio. Compared to traditional cement concrete pavements, the optimized EAC, while maintaining mechanical properties, exhibits superior SRNR performance, providing a valuable reference for the construction of high SRNR cement concrete pavements.

Keywords: road engineering; exposed aggregate concrete; surface texture characteristics; skid resistance; noise reduction; mechanical properties; orthogonal experimentation

1. Introduction

To ensure adequate skid resistance of cement concrete pavements, conventional surface treatments such as grooving and brooming are commonly employed; however, these traditional methods exhibit functional deficiencies, including rapid skid resistance degradation and excessive traffic noise, thereby compromising driving safety and comfort [1,2]. Relevant studies indicate that when the sideways force coefficient (SFC) of cement pavements drops below 45, the incidence of rainy-day traffic accidents increases by over 30% compared to clear weather conditions [3,4], while tire/pavement noise has emerged as a critical control factor in environmental noise management [5–7]. Against this background,

as requirements for driving safety enhancement and road traffic noise reduction continue to escalate [8,9], SRNR cement concrete pavements have gained increasing attention in road engineering. Among these, EAC pavements, owing to their asphalt-like surface macrotexture, demonstrate superior SRNR performance, making them one of the preferred solutions for high SRNR performance cement concrete pavements.

Cai et al. [10,11] analyzed the relationship between the mortar layer thickness and the exposed aggregate depth in EAC, and established a predictive model between the mortar layer thickness and the mortar sweeping amount. It was found that as the mortar layer thickness increased, the exposed aggregate depth continuously decreased, while the mortar removal volume increased with the elevated sum of the mortar layer thickness and the exposed aggregate depth. Wei [12] compared the noise reduction performance of asphalt pavements, EAC pavements, and conventional cement concrete pavements using the tire drop test method, demonstrating that EAC pavements exhibited the highest noise attenuation efficiency. Kim et al. [13] experimentally demonstrated that under identical surface degradation conditions, EAC incorporating 8 mm and 10 mm coarse aggregates exhibited 43% and 36% lower wear depths, respectively, compared to grooved cement concrete. Yan et al. [14] compared the road performance of EAC with different sand ratios and different coarse aggregate gradations, and the best road performance was achieved when the sand ratio was 20% and the ratio of 4.75~9.5 mm to 9.5~13.2 mm coarse aggregate was 2:1. Chhay et al. [15] constructed a 3D model of EAC surface texture through digital image processing technology, confirming that this technology can effectively characterize the surface texture of EAC. Yang et al. [16] investigated the long-term skid resistance of EAC, concluding that the larger the particle size of the aggregate, the better the long-term skid resistance. Chen et al. [17] investigated the surface treatment process for EAC, establishing that the optimal spraying time and dosage of the exposed aggregate agent range from 30~90 min and 200~250 g/m², respectively. Hu et al. [18] investigated the effects of macro texture depth (MTD) on surface roughness and skid resistance performance of EAC, revealing that EAC achieves optimal skid resistance when MTD is controlled within the 0.72~0.80 mm range. Rith et al. [19] employed pavement texture parameters and aggregate polishing resistance to characterize the long-term skid resistance performance of EAC and established a predictive model for its long-term skid resistance. Han et al. [20] conducted noise spectrum analysis and found that EAC pavements exhibit a significant reduction in noise value above 1.25 kHz compared to plain concrete pavements. Zhao et al. [21] investigated the relationship between different coarse aggregate exposure postures and the long-term skid resistance of EAC, pointing out that the coarse aggregate with flat-lying or inclined exposure has better long-term skid resistance. Hablovicova et al. [22] utilized the Close Proximity method (CPX) to compare the noise data of EAC (maximum aggregate size: 8 mm) and SMA-11 asphalt pavements at various time intervals, revealing that both pavement types exhibited similar noise levels and followed a logarithmic variation pattern over their lifetime.

In summary, current research on EAC has predominantly focused on the impacts of surface treatment techniques, texture characteristics, coarse aggregate composition, and exposure patterns on its SRNR performance, while studies concerning the formation mechanisms of surface texture characteristics and optimized mix design that comprehensively considers factors including water–binder ratio, sand ratio, coarse aggregate composition, and dosage have remained comparatively scarce in existing literature. To address this gap, the present study employed an orthogonal experimental method to prepare EAC samples with varying mix proportions and systematically investigated the effects of four mix design parameters—water–binder ratio, sand ratio, coarse aggregate volume ratio, and proportion of aggregates > 9.5 mm—on surface texture characteristics, SRNR performance,

and mechanical properties. Based on these analyses, the optimal mix proportions for EAC with enhanced SRNR capabilities are proposed, thereby providing a scientific basis for the application of EAC with high SRNR performance in pavement engineering.

2. Materials and Methods

2.1. Raw Materials

The experimental materials included ordinary Portland cement (OPC) of grade P·O 42.5, with a density of 3110 kg/m^3 and a specific surface area of $3360 \text{ cm}^2/\text{g}$. The fine aggregate was river sand classified as Zone II medium sand with a maximum particle size of 2.36 mm, a fineness modulus of 2.48, and an apparent density of 2655 kg/m^3 . The coarse aggregate comprised basalt gravel in two size fractions: 4.75–9.5 mm and 9.5–16 mm, with an apparent density of 2930 kg/m^3 , a crushing value of 10.7%, an elongated/flaky particle content of 1.2%, and a Los Angeles abrasion value of 11.8%. The exposed aggregate agent employed was MT-308, a hydroxy carboxylic acid-based organic compound with a pH of 8.5 that exhibits a black, viscous liquid appearance. Its mechanism involves penetrating the concrete matrix to encapsulate cement particles, thereby inhibiting their hydration reaction with water. This action effectively delays the final setting time of the concrete surface by 12–24 h. Tap water was used for mixing. All materials satisfied the technical requirements stipulated in the Technical Guidelines for Construction of Highway Cement Concrete Pavements (JTG/T F30-2014) [23], confirming their suitability for the preparation of EAC.

2.2. Orthogonal Experimental Design Scheme

The surface texture characteristics of EAC are intrinsically linked to coarse aggregate content and gradation. Therefore, two mix design parameters—coarse aggregate volume ratio and proportion of aggregates $> 9.5 \text{ mm}$ —were selected to reflect coarse aggregate dosage and gradation, alongside two conventional parameters (water–binder ratio and sand ratio), forming four factors in the orthogonal experimental design. Based on existing research on the mix proportion design of EAC [24–28] and technical requirements specified in the Specification for Mix Proportion Design of Ordinary Concrete (JGJ 55-2011) [29], the appropriate ranges for each parameter were determined as follows: water–binder ratio 0.40–0.46, sand ratio 28–34%, coarse aggregate volume ratio 40–44%, and proportion of aggregates $> 9.5 \text{ mm}$ 30–70%. After comprehensive consideration of factors including specimen fabrication costs, number of experiments, parameter range settings, and the three-level experimental design which facilitates inflection point analysis, each factor was assigned three levels based on the aforementioned ranges, and an $L_9(3^4)$ orthogonal array was adopted to organize laboratory tests for physical and mechanical properties. The factor-level assignments are detailed in Table 1, with the Orthogonal test scheme provided in Table 2. Here, Factors A, B, C, and D correspond to water–binder ratio, sand ratio, coarse aggregate volume ratio, and proportion of aggregates $> 9.5 \text{ mm}$.

To rationally analyze the surface texture characteristics, SRNR performance, and mechanical properties of EAC, two indicators—exposed aggregate depth and exposed aggregate area ratio—were used to characterize the surface texture. The pendulum value and noise value were employed to evaluate the SRNR performance, respectively. The 28-day compressive and flexural strengths were used to assess the mechanical properties of EAC. Therefore, a total of six performance indicators were tested to optimize the EAC mix proportion. To minimize potential errors during EAC preparation and testing, three parallel specimens were fabricated for each mix proportion in the orthogonal experiment, resulting in 27 specimens in total. The corresponding experimental results for each group were determined as the average value of the three specimens. The experimental error

was controlled using the absolute error percentage of three parallel data relative to the median value. For exposed aggregate depth and area ratio, an absolute error threshold of 5% was established: when the error exceeded 5%, the median value was retained as the valid measurement if only one datum exceeded the threshold, whereas the entire specimen group was invalidated if both data exceeded the threshold. Similarly, for the pendulum value and noise value, the absolute error tolerance was adjusted to 3%, while the tolerance for 28-day compressive and flexural strength tests was set at 15%.

Table 1. Factors, levels.

Levels	Factors			
	A Water–Binder Ratio	B Sand Ratio/%	C Coarse Aggregate Volume Ratio/%	D Proportion of Aggregates >9.5 mm/%
1	0.40	28	40	30
2	0.43	31	42	50
3	0.46	34	44	70

Table 2. $L_9(3^4)$ Orthogonal test scheme.

Test Number	Factors			
	A	B	C	D
1	0.40	28	40	30
2	0.40	31	42	50
3	0.40	34	44	70
4	0.43	28	42	70
5	0.43	31	44	30
6	0.43	34	40	50
7	0.46	28	44	50
8	0.46	31	40	70
9	0.46	34	42	30

2.3. Specimen Preparation and Test Methods

2.3.1. Specimen Preparation

EAC slab specimens were prepared using 300 mm × 300 mm × 50 mm rutting plate molds through the following procedures: (1) fresh concrete was batched, mixed, cast into molds, vibrated, and screeded, then left undisturbed at ambient temperature; (2) after surface water film disappearance, exposed aggregate agent was uniformly sprayed onto specimen surfaces followed by plastic film covering to prevent evaporation; (3) specimens were cured in a constant temperature chamber maintained at 20 °C and relative humidity >90%; (4) following a 18-h curing period, pressure water jets and brushes were applied to expose coarse aggregates; (5) post-washing specimens were transferred to standard curing chambers at 20 ± 5 °C and relative humidity > 95% for moisture retention until designated testing ages for physico-mechanical evaluations. The completed EAC slabs are illustrated in Figure 1.



Figure 1. EAC slab specimen: (a) Before brushing. (b) After brushing.

2.3.2. Exposed Aggregate Depth Test

Exposed aggregate depth refers to the height of exposed coarse aggregates on the surface of EAC, serving as a critical indicator for evaluating the exposing effect. According to relevant literature [30], the optimal exposed aggregate depth should be controlled within the range of 2~3 mm. The disc method [17] was employed to measure the exposed aggregate depth. During testing, a transparent plexiglass disc with circular holes was placed on the surface of the EAC slab. A measurement probe was vertically inserted into the hole. If the reading on the probe exceeded the disc thickness, the measurement was considered valid. Conversely, if the reading was less than the disk thickness, it indicated that the measurement point was located at a protrusion of coarse aggregates, rendering the data invalid. Based on this method, ten random test positions were uniformly selected across the disk surface, with the mean of these ten measurements serving as a single valid measurement. The aforementioned steps were repeated five times to obtain five valid measurements, and the coefficient of variation (CV) of these five values was calculated. When CV exceeded 0.05, the disk was repositioned for additional sampling; otherwise, the exposed aggregate depth was obtained by subtracting the disk thickness from the valid mean measurement. The specific testing setup is illustrated in Figure 2.



Figure 2. Exposed aggregate depth test.

2.3.3. Exposed Aggregate Area Ratio Analysis

The exposed aggregate area ratio represents the percentage of the unit surface area of EAC occupied by exposed coarse aggregates, serving as a critical indicator for characterizing the surface texture of EAC. According to the Technical Guidelines for Construction of Highway Cement Concrete Pavements (JTG/T F30-2014) [23], the exposed aggregate area ratio on EAC surfaces should, optimally, be controlled between 65% and 75%.

Given the distinct color contrast between the exposed black basalt coarse aggregates and the surrounding off-white cement mortar on the EAC slab surface, the exposed aggregate area ratio can be obtained using digital image processing techniques [31] via MATLAB R2022b software. The specific processing steps are as follows: (1) The captured surface image of EAC slab was subjected to grayscale conversion, yielding the grayscale frequency distribution histogram shown in Figure 3; (2) Considering the inevitable presence of residual cement paste stains on exposed aggregate surfaces, causing localized off-white discoloration, a skewed normal distribution was employed to characterize the grayscale frequency distribution of coarse aggregates, while a normal distribution was adopted for the cement mortar due to its relatively minor color contamination. Based on the above analysis, a weighted combination function of skew-normal and normal distributions was utilized to fit the grayscale frequency data of EAC slab image, with the specific mathematical formulation detailed in Equation (1) and the fitting curve results demonstrated in Figure 3; (3) In the fitting results, the integral area under the skew-normal distribution curve within the 0~255 grayscale range was calculated as the exposed aggregate area ratio

of the EAC slab. Figure 4 shows the pre- and post-grayscale processing results of a typical exposed aggregate slab surface image.

$$f(x) = \frac{2a}{\sigma_1} \phi\left(\frac{y - \mu_1}{\sigma_1}\right) \Phi\left(\lambda \cdot \frac{y - \mu_1}{\sigma_1}\right) + \frac{b}{\sqrt{2\pi}\sigma_2} \exp\left[-\frac{1}{2}\left(\frac{x - \mu_2}{\sigma_2}\right)^2\right] \quad (1)$$

where $f(x)$ represents the grayscale value; $\phi(\cdot)$ denotes the probability density function of the standard normal distribution, while $\Phi(\cdot)$ denotes its cumulative distribution function; parameters μ_1 , σ_1 and μ_2 , σ_2 correspond to the mean and standard deviation of the skew-normal distribution and normal distribution, respectively; λ serves as the skewness parameter for the skew-normal distribution; and a and b represent the weighting coefficients for the two distributions.

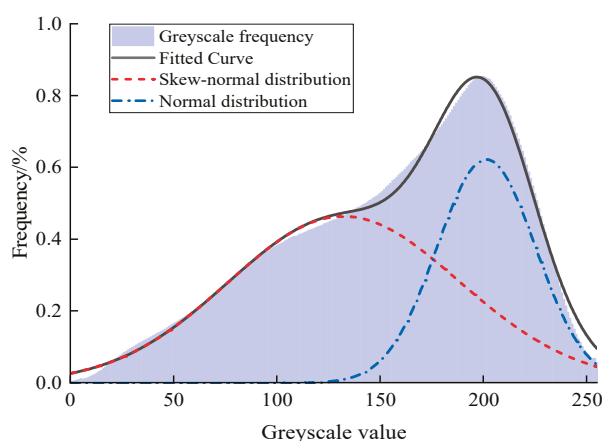


Figure 3. Greyscale frequency distribution fitting results.

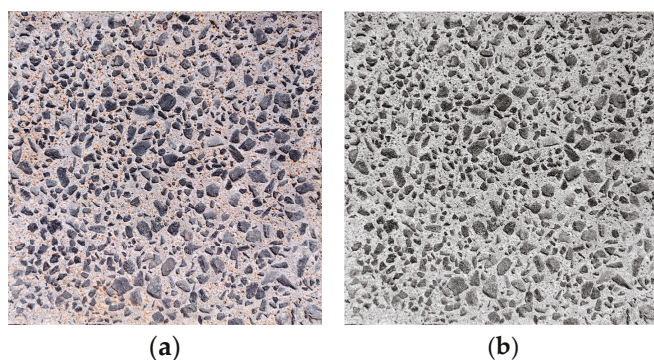


Figure 4. EAC slab images before and after grayscale image processing: (a) Before grayscale processing. (b) After grayscale processing.

2.3.4. Skid Resistance Performance Test

The skid resistance performance of the EAC slab surface was evaluated using the pendulum friction tester method (T 0964) specified in the Field Test Methods of Highway Subgrade and Pavement (JTG 3450-2019) [32], with the British Pendulum Number (BPN) serving as the indicator of skid resistance. Testing was conducted at 25 °C ambient temperature with surface moistening achieved through water spray bottle application. The test result was derived from the average of five consecutive measurements, followed by temperature correction as prescribed by the standard. The specific testing configuration is illustrated in Figure 5.



Figure 5. Pendulum value test.

2.3.5. Noise Reduction Performance Test

Owing to the dimensional constraints of the EAC slab specimens, the indoor tire drop method [33] was implemented to simulate traffic-induced noise on the slab surface. The DH5922D dynamic signal acquisition and analysis system was employed to capture the impact noise generated between the tire and the EAC slab. The test tire used was a standard passenger car tire (205/55 R16) inflated to a pressure of 0.22 MPa. The ambient temperature during testing was 26 °C, and the tire was dropped from a height of 65 cm above the EAC slab surface. Noise sensors were positioned symmetrically on the same side of the slab, each located 100 cm horizontally and 70 cm vertically from the slab center, forming a 45° angle with the line passing through the slab center and perpendicular to the tire's direction of travel. During testing, the ambient background noise level was maintained below 30 dB to minimize interference. The specific test procedures are as follows: (1) connecting the noise sensors and initiating noise data acquisition; (2) releasing the tire from an elevated platform to generate impact noise with the EAC slab and acquiring the noise frequency spectrum; (3) recording maximum noise peaks during tire/surface interaction; (4) repeating the sequence five times, and calculating the arithmetic mean of measurements as the noise value of the EAC slab. The entire noise testing system configuration is illustrated in Figure 6.



Figure 6. Noise testing system.

2.3.6. Mechanical Properties Test

The mechanical properties of the concrete were tested in accordance with the 28-day compressive and flexural strength test methods (T 0553 and T 0558) for concrete specified in the Testing Methods of Cement and Concrete for Highway Engineering (JTG 3420-2020) [34]. Specifically, the 28-day compressive strength test was conducted using a DYE-2000 digital compression testing machine, as shown in Figure 7a, while the 28-day flexural strength test was performed using a WE-300B universal material testing machine (Shuangniu Building Materials Instrument and Equipment Factory, Wuxi, China), as illustrated in Figure 7b.

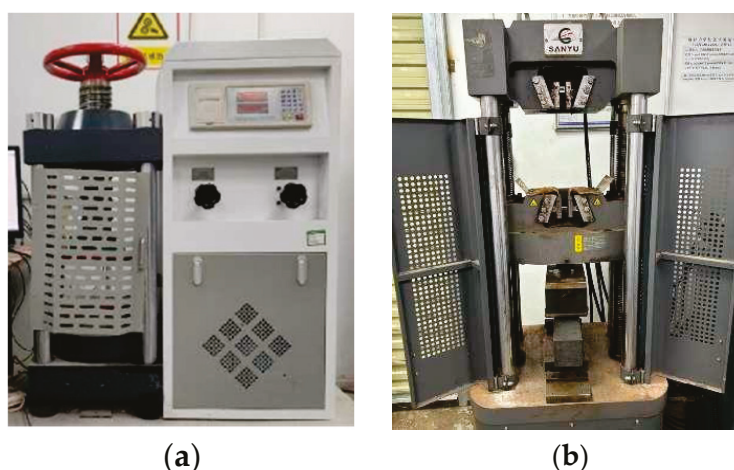


Figure 7. Twenty-eight-day compressive and flexural strength test: (a) Compressive strength tester. (b) Flexural strength tester.

3. Test Results

3.1. Analysis of Orthogonal Test Results

The surface texture characteristics, SRNR performance, and mechanical properties indicators of the EAC slab specimens prepared through orthogonal experimental design were tested. The test results are presented in Table 3, and the corresponding range analysis results are shown in Table 4.

By comparing the range values of the four factors, the influence degrees of each mix design parameter on the six performance indicators of EAC can be ranked from high to low as follows: exposed aggregate depth $D > B > C > A$; exposed aggregate area ratio $D > C > B > A$; pendulum value $D > C > A > B$; noise value $D > A > B > C$; 28-day compressive strength $A > B > C > D$; and 28-day flexural strength $B > D > A > C$. The above results indicate that, for the surface texture characteristics of EAC, the proportion of aggregates > 9.5 mm has the most significant impact on both exposed aggregate depth and area ratio, while the water–binder ratio exerts the minimal influence. Regarding skid resistance performance, the proportion of aggregates > 9.5 mm remains the most influential factor. Both the water–binder ratio and sand ratio contribute marginally to skid resistance, with their range values being less than 2 BPN, indicating a substantial gap compared to the impact of other factors. For noise reduction performance, the proportion of aggregates > 9.5 mm is still the most significant factor, whereas the coarse aggregate volume ratio has the least impact on noise value. In terms of mechanical properties, the water–binder ratio and sand ratio are the most significant factors affecting the 28-day compressive strength and 28-day flexural strength, respectively. The coarse aggregate volume ratio has a negligible impact on the 28-day flexural strength, with a range of only 0.08 MPa.

Table 3. Results of the orthogonal test.

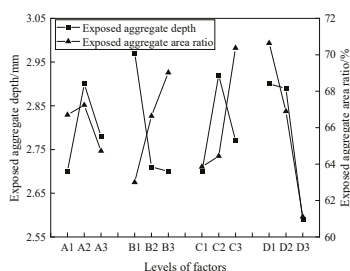
Test Number	Exposed Aggregate Depth/mm	Exposed Aggregate Area Ratio/%	Pendulum Value/BPN	Noise Value/dB	28-day Compressive Strength/MPa	28-day Flexural Strength/MPa
1	2.88	65.53	70.00	82.44	47.42	5.13
2	2.84	66.03	74.67	82.37	50.69	5.53
3	2.38	68.54	73.00	85.38	51.86	5.70
4	3.00	57.13	79.67	81.84	44.57	5.13
5	2.89	76.23	70.00	82.02	48.29	5.20
6	2.81	68.34	71.67	81.66	48.86	5.27
7	3.03	66.33	71.00	81.59	42.27	4.90
8	2.40	57.67	77.67	84.28	38.97	5.53
9	2.92	70.17	74.33	82.82	44.15	5.03

Table 4. Range analysis of an orthogonal test.

Test Indicators		Factors			
		A	B	C	D
Exposed aggregate depth/mm	k_1	2.70	2.97	2.70	2.90
	k_2	2.90	2.71	2.92	2.89
	k_3	2.78	2.70	2.77	2.59
	range	0.20	0.27	0.22	0.31
Exposed aggregate area ratio/%	k_1	66.70	63.00	63.85	70.64
	k_2	67.24	66.64	64.44	66.90
	k_3	64.72	69.02	70.37	61.11
	range	2.51	6.02	6.52	9.53
Pendulum value/BPN	k_1	72.56	73.56	73.11	71.44
	k_2	73.78	74.11	76.22	72.44
	k_3	74.33	73.00	71.33	76.78
	range	1.78	1.11	4.89	5.33
Noise value/dB	k_1	83.39	81.96	82.79	82.43
	k_2	81.84	82.89	82.34	81.87
	k_3	82.90	83.29	83.00	83.83
	range	1.55	1.33	0.66	1.96
28-day compressive strength/MPa	k_1	49.99	44.75	45.08	46.62
	k_2	47.24	45.98	46.47	47.27
	k_3	41.80	48.29	47.47	45.13
	range	8.19	3.54	2.39	2.14
28-day flexural strength/MPa	k_1	5.45	5.05	5.31	5.12
	k_2	5.20	5.42	5.23	5.23
	k_3	5.16	5.33	5.27	5.46
	range	0.30	0.37	0.08	0.33

3.2. Variation Patterns of Various Factors on Surface Texture Characteristics

The surface texture characteristics of EAC are critical to its SRNR performance. According to the Technical Guidelines for Construction of Highway Cement Concrete Pavements (JTG/T F30-2014) [23] and relevant literature [30], the exposed aggregate depth of EAC should be controlled between 2 and 3 mm, and the exposed aggregate area ratio should be maintained between 65 and 75%. Based on the range analysis results of the orthogonal test shown in Table 4, the variation patterns of the average exposed aggregate depth and area ratio under different factor levels are illustrated in Figure 8.

**Figure 8.** The variation patterns of exposed aggregate depth and area ratio.

As shown in Figure 8, as the water–binder ratio increases, both the exposed aggregate depth and the exposed aggregate area ratio first increase and then decrease. When the water–binder ratio increases from 0.40 to 0.43, the elevation of the water–binder ratio prolongs the setting time of the surface cement mortar. Under identical brushing timing conditions, the EAC surface with a higher water–binder ratio exhibits greater amounts of unset cement mortar, leading to greater exposed aggregate depth and area ratio after brushing. When the water–binder ratio further increases from 0.43 to 0.46, the enhanced fluidity of the EAC mixture causes internal cement mortar to rise to the surface during vibration, while coarse aggregates sink. This increases the surface mortar layer thickness, making cement mortar the primary component within the influence depth of the exposed aggregate agent, rather than the coarse aggregate. Consequently, both the exposed aggregate depth and area ratio decrease after brushing.

As the sand ratio increases, the exposed aggregate depth gradually decreases, while the exposed aggregate area ratio gradually increases. When the sand ratio increases from 28% to 34%, the fine aggregates progressively fill the voids between coarse aggregates, enhancing the fluidity and dispersion of coarse aggregates during EAC vibration. This reduces the convex peak structures formed by the mutual extrusion of coarse aggregates at the surface, thereby gradually lowering and stabilizing the exposed aggregate depth. Simultaneously, the increased sand ratio enlarges the total specific surface area of aggregates, reducing concrete fluidity. During vibration, coarse aggregates become less prone to sinking, leading to a gradual increase in the exposed aggregate area ratio.

As the coarse aggregate volume ratio increases, the exposed aggregate depth first increases and then decreases, while the exposed aggregate area ratio gradually increases. When the coarse aggregate volume ratio increases from 40% to 42%, the proportion of coarse and fine aggregates in the concrete mixture gradually increases, while the cement paste proportion decreases. This reduces the cement paste's ability to coat the aggregates. The reduced concrete fluidity decreases the surface mortar thickness, increasing the exposed aggregate depth and slightly raising the exposed aggregate area ratio by 0.59%. When the coarse aggregate volume ratio increases from 42% to 44%, the cement paste proportion further decreases, leaving insufficient paste to coat all aggregates. Both the cement paste deficiency and aggregate surface adsorption reduce the effective penetration depth of the exposed aggregate agent, causing a decrease in exposed aggregate depth. Concurrently, the further reduced concrete fluidity impedes coarse aggregate particles from sinking during vibration, leading to their accumulation on the surface and a significant increase in the exposed aggregate area ratio, reaching 5.93%.

As the proportion of aggregates >9.5 increases, both exposed aggregate depth and exposed aggregate area ratio exhibit decreasing trends. Specifically, the minimum exposed aggregate depth drops by 10.7% compared to its maximum value, while the exposed aggregate area ratio decreases by 9.53%. Among all factors, the proportion of aggregates >9.5 exerts the most significant influence on surface texture characteristics. As this proportion increases from 30% to 70%, the proportion of large-diameter particles in the coarse aggregates increases, the gaps between the coarse aggregates enlarge, and the specific surface area of the aggregates decreases. These changes enhance the internal mortar flow capacity of the EAC, making it easier for the mortar to float to the surface during concrete vibration. Meanwhile, large-particle coarse aggregates are more likely to sink, resulting in a significant reduction in both the exposed aggregate depth and the exposed aggregate area ratio.

3.3. Variation Patterns of Various Factors on SRNR Performance

As indicated in the previous analysis, various factors have varying impacts on the surface texture characteristics of EAC, which directly correlate with the SRNR performance

of EAC surfaces. Therefore, it is necessary to analyze the influence of each factor on the SRNR performance of EAC surfaces based on their effects on the surface texture characteristics. Figure 9 illustrates the variation patterns of the average pendulum value and average noise value of EAC surfaces under different factor levels.

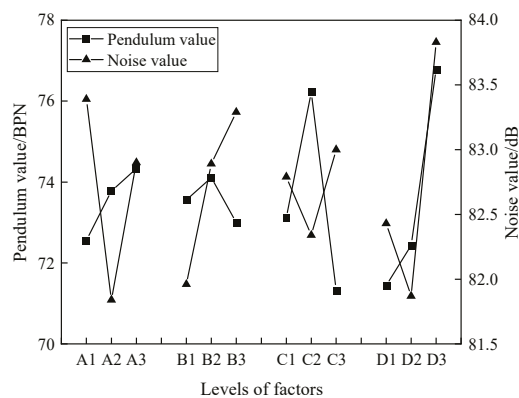


Figure 9. The variation patterns of the pendulum value and noise value.

3.3.1. Variation Patterns of Various Factors on Skid Resistance Performance

As shown in Figure 9, as the water–binder ratio increases, the pendulum value gradually rises; with the increase in the sand ratio, the pendulum value first increases and then decreases. For both water–binder ratio and sand ratio, the maximum value does not exceed 2.5% compared to the minimum value. This indicates that the influence of water–binder ratio and sand ratio on the skid resistance performance of EAC surfaces is not significant.

As the coarse aggregate volume ratio increases, the pendulum value first increases and then decreases. When the coarse aggregate volume ratio increases from 40% to 42%, the exposed aggregate area ratio changes slightly, only increasing by a small margin of 0.59%, while the exposed aggregate depth increases by 0.22 mm. The rise in pendulum value is primarily attributed to the increased exposed height of coarse aggregates on the EAC surface, enhancing the frictional effect of the coarse aggregates and thereby improving surface skid resistance performance. When the coarse aggregate volume ratio increases from 42% to 44%, the exposed aggregate area ratio increases by 5.93%, while the exposed aggregate depth decreases by 0.15 mm. According to research by the International Road Federation (IRF), the skid resistance performance between tires and pavement primarily depends on the pavement surface micro-texture under low-speed driving conditions [35]. On EAC surfaces, the mortar possesses richer micro-textures. The pendulum friction tester simulates low-speed vehicle driving; therefore, the primary reason for the decrease in pendulum value at this stage is that the increased coarse aggregate area ratio obstructs contact between the pendulum friction tester's rubber slider and the cement mortar, thereby reducing the EAC surface skid resistance performance.

As the proportion of aggregates > 9.5 mm increases, the pendulum value gradually increases. When the proportion of aggregates > 9.5 mm increases from 30% to 50%, the exposed aggregate depth only decreases by 0.01 mm, and the exposed aggregate area ratio decreases by 3.74%, while the pendulum value only increases by 1.00 BPN. This indicates that when the exposed aggregate area ratio is from 66.90% to 70.64%, the impact of the exposed aggregate area ratio on the pendulum value is relatively small. When the proportion of aggregates > 9.5 mm increases from 50% to 70%, the exposed aggregate depth decreases by 0.30 mm, and the exposed aggregate area ratio decreases by 5.79%, while the pendulum value increases by 4.34 BPN. This indicates that when the exposed aggregate area ratio is from 61.11% to 66.90%, the impact of the exposed aggregate area ratio on the pendulum value is relatively large. Although the frictional resistance of the coarse

aggregate decreases, the contact area between the rubber slider of the pendulum friction tester and the cement mortar continuously increases, enhancing the surface skid resistance of the EAC.

3.3.2. Variation Patterns of Various Factors on Noise Reduction Performance

As shown in Figure 9, as the water–binder ratio increases, the noise value first decreases and then increases. This phenomenon is mainly due to the fact that when the water–binder ratio is 0.43, the corresponding exposed aggregate depth is the largest, and there are the most voids between the tire and the EAC surface. Relevant studies have shown that the voids between the tire and the pavement can effectively reduce the pumping noise generated when the tire contacts the pavement surface [36,37]. Therefore, when the water–binder ratio is 0.43, the noise value of the EAC surface is the lowest.

As the sand ratio increases, the noise value gradually rises, with the maximum value increasing by 1.33 dB compared to the minimum value. When the sand ratio increases from 28% to 31%, the exposed aggregate depth decreases by 0.26 mm, and the voids between the tire and the surface decrease, leading to an increase in the noise value. When the sand ratio increases from 31% to 34%, the exposed aggregate depth changes little, decreasing by only 0.01 mm; however, the exposed aggregate area ratio increases by 2.38%. At this point, the increase in the noise value is mainly due to the increase in the exposed aggregate area ratio, which results in the accumulation of more coarse aggregates on the EAC surface. This accumulation reduces the voids between the coarse aggregates, enhancing the air pumping effect between the tire and the pavement, as well as the vibration noise of the tire itself.

As the coarse aggregate volume ratio increases, the noise value first decreases and then increases. When the coarse aggregate volume ratio increases from 40% to 42%, the exposed aggregate area ratio only increases by 0.59%, while the exposed aggregate depth increases by 0.22 mm, resulting in a decrease in the noise value. When the coarse aggregate volume ratio increases from 42% to 44%, the exposed aggregate area ratio increases by 5.93%, while the exposed aggregate depth decreases by 0.15 mm. The increase in coarse aggregates on the EAC surface, coupled with the decrease in exposed aggregate depth, leads to the blockage of air escape channels between the tire and the surface, thereby increasing the noise value.

As the proportion of aggregates > 9.5 mm increases, the noise value first decreases and then increases. When the proportion of aggregates > 9.5 mm increases from 30% to 50%, the exposed aggregate area ratio decreases by 3.74%, while the exposed aggregate depth only decreases by 0.01 mm, resulting in a decrease in the noise value. When the proportion of aggregates > 9.5 mm increases from 50% to 70%, the exposed aggregate area ratio decreases by 5.79%, and the exposed aggregate depth decreases by 0.30 mm. At this point, the noise value increases by 2.96 dB compared to when the proportion of aggregates > 9.5 mm is 50%. This indicates that, compared to the exposed aggregate area ratio, the exposed aggregate depth has a greater impact on the noise value.

3.4. Variation Patterns of Various Factors on Mechanical Properties

The mechanical properties of EAC are important technical indicators that determine its application scenarios. The strength of EAC designed in this study should meet the C40 compressive strength grade (≥ 40 MPa) and the requirements for concrete flexural strength (≥ 5 MPa) under heavy and above traffic load grades specified in the Specifications for Design of Highway Cement Concrete Pavement (JTG D40-2011) [38]. The variation patterns of average 28-day compressive and flexural strength under different factor levels are shown in Figure 10.

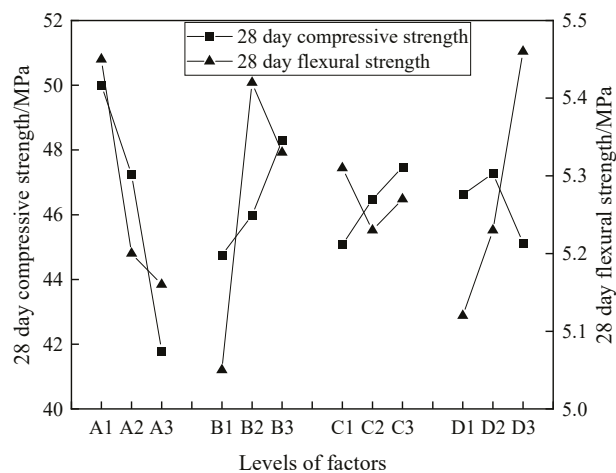


Figure 10. The variation patterns of 28-day compressive and flexural strength.

As shown in Figure 10, as the water–binder ratio increases, both the 28-day compressive and flexural strengths gradually decrease. This is mainly because an increase in the water–binder ratio affects the hydration reaction of cement, and excess water easily forms blisters or pores after concrete hardening, resulting in excessive pores inside the concrete, which, in turn, reduces the strength of the concrete.

As the sand ratio increases, the 28-day compressive strength gradually increases, while the 28-day flexural strength first increases and then decreases. When the sand ratio increases from 28% to 31%, the fine aggregates continuously fill the voids between the coarse aggregates, making the concrete structure denser and the stress distribution between the aggregates more uniform, thereby improving both the compressive and flexural strengths. When the sand ratio increases from 31% to 34%, the interlocking effect of the microscopic engagement structure between the fine aggregates is further enhanced, increasing the compressive strength. However, at this point, an increase in the sand ratio increases the specific surface area of the aggregates, and insufficient coating of the aggregates by the cement paste leads to a decrease in the flexural strength of the concrete interface.

As the coarse aggregate volume ratio increases, the 28-day compressive strength gradually increases, while the 28-day flexural strength first decreases and then increases. For compressive strength, an increase in the coarse aggregate volume ratio increases the “point-to-point” contact between coarse aggregates, improving the interlocking force between them, and thus gradually increasing the compressive strength. For flexural strength, when the coarse aggregate volume ratio is 40%, the flexural strength reaches a maximum value of 5.31 MPa, which is only an increase of 1.5% compared to the minimum value, indicating that the coarse aggregate volume ratio has a small impact on the 28-day flexural strength.

As the proportion of aggregates > 9.5 mm increases, the 28-day compressive strength first increases and then decreases, while the 28-day flexural strength gradually increases. For compressive strength, when the proportion of aggregates > 9.5 mm is 50%, the aggregates form a skeleton-dense structure, resulting in the highest overall compressive strength of the concrete structure. For flexural strength, the larger the proportion of aggregates > 9.5 mm, the smaller the specific surface area of the aggregates, the higher the proportion of cement mortar at the cross-section, and the smaller the area of the interface weak zone composed of mortar and coarse aggregates, thus resulting in higher flexural strength.

4. Discussion

4.1. Optimisation of Mix Proportions

Based on the results of the above orthogonal test analysis, aiming at maximizing the pendulum value, minimizing the noise value, maximizing the 28-day compressive strength, and maximizing the 28-day flexural strength, the optimal level combinations of the four mix design factors (water–binder ratio, sand ratio, coarse aggregate volume ratio, and proportion of aggregates > 9.5 mm) for the SRNR performance and mechanical properties of EAC can be obtained. The results are shown in Table 5.

Table 5. Optimized combination results.

Test Indicators	Factors			
	A	B	C	D
Pendulum value	0.46	31	42	70
Noise value	0.43	28	42	50
28-day compressive strength	0.40	34	44	50
28-day flexural strength	0.40	31	40	70

The optimal mix proportions of EAC should be based on meeting the basic requirements of concrete strength and surface texture characteristics, while pursuing higher SRNR performance. When the water–binder ratio is 0.46, the EAC tends to exhibit reduced exposed aggregate depth and area ratio due to excessive mortar layer thickness; therefore, the water–binder ratio is chosen as 0.43. When the sand ratio is 28%, the fine aggregates are insufficient to fill the voids between the coarse aggregates, resulting in interlocking between the coarse aggregates, which is not easily dispersed. The coarse aggregates are easily unevenly distributed in the exposed height on the surface of EAC, and the exposed aggregate area ratio is reduced. Therefore, the sand ratio is selected as the optimal level of 31% for the pendulum value and 28-day flexural strength. The coarse aggregate volume ratio is selected as the optimal level of 42% for the pendulum value and noise value. The proportion of aggregates > 9.5 mm has the greatest impact on the surface texture characteristics of EAC. When the proportion of aggregates > 9.5 mm is 70%, both the exposed aggregate depth and the exposed aggregate area ratio will be significantly reduced to low levels. Although the skid resistance performance may improve at this stage due to the increased mortar area ratio, the reduction in coarse aggregates on the surface simultaneously compromises its abrasion resistance. Therefore, in order to ensure that the EAC surface has sufficient exposed aggregate depth and exposed aggregate area ratio, the proportion of aggregates > 9.5 mm is selected as 50%.

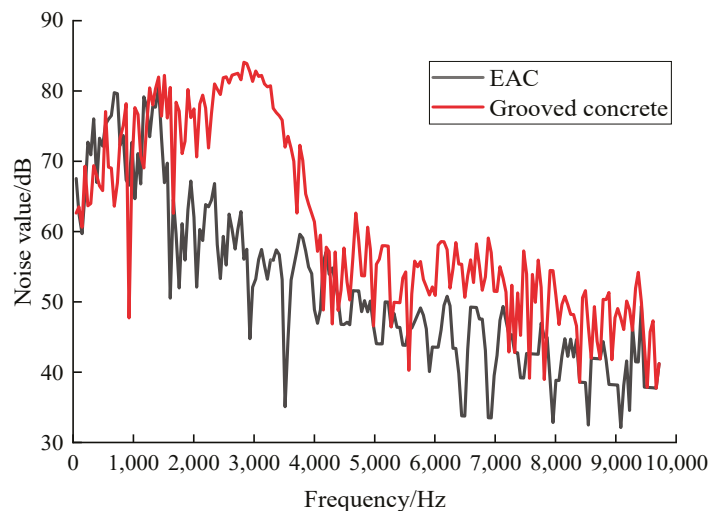
In summary, the optimal mix proportions for EAC with high SRNR performance are determined as follows: water–binder ratio of 0.43, sand ratio of 31%, coarse aggregate volume ratio of 42%, and proportion of aggregates > 9.5 mm of 50%.

4.2. Verification of Optimal Mix Proportions

The optimal mix proportions were selected to fabricate EAC slab specimens, and their surface texture characteristics, SRNR performance, mechanical properties, and workability indicators were experimentally verified. Meanwhile, to compare the high SRNR performance of EAC with the optimal mix proportions, grooved cement concrete slab specimens with a texture width of 6 mm, depth of 3 mm, and texture spacing of 15 mm were fabricated as a control group. The corresponding test results are shown in Table 6. A spectral analysis comparison of the A-weighted tire/surface noise between EAC and grooved concrete was conducted, and the results are shown in Figure 11.

Table 6. The optimal mix proportions verification and comparison.

Test Indicators	Technical Requirements	EAC	Grooved Concrete
Exposed aggregate depth/mm	2~3	2.89	/
Exposed aggregate area ratio/%	65~75	68.75	/
Pendulum value/BPN	/	77.56	58.98
Noise value/dB	/	81.53	88.01
28-day compressive strength/MPa	≥ 40	47.81	43.23
28-day flexural strength/MPa	≥ 5	5.35	5.12
Slump/mm	10~30	14	23

**Figure 11.** Tire/surface noise spectrum of grooved concrete and EAC.

As can be seen from Table 6 and Figure 11, compared with grooved concrete, the pendulum value of EAC increased by 31.5%, and the noise value decreased by 6.48 dB. The frequency range of noise reduction was mainly concentrated between 1.5 and 4.0 kHz, which falls within the mid-to-high frequency range from 1.0 to 5.0 kHz that is more sensitive to the human ear [39]. This indicates that the optimized EAC, due to its rich surface texture, has superior SRNR performance compared to ordinary concrete, and can serve as a reference for engineering applications.

4.3. Correlation Analysis of SRNR Performance

Based on the analysis of orthogonal test results, it is not difficult to find that there is a certain correlation between the surface texture characteristics of EAC and its SRNR performance. Therefore, a correlation analysis was conducted on the orthogonal test data of nine groups of 27 EAC slab specimens, including exposed aggregate depth, exposed aggregate area ratio, pendulum value, and noise value. The analysis results are shown in Figures 12 and 13, respectively. The results showed that the coefficient of determination R^2 between noise value and exposed aggregate depth, as well as between pendulum value and exposed aggregate area ratio, both exceeded 0.64, indicating that both relationships exhibited high correlations.

As can be seen from Figure 12, as the exposed aggregate depth increases, the noise value gradually decreases and stabilizes. Based on this variation characteristic of noise value, an inverse exponential function was used for fitting, and the coefficient of determination R^2 was found to be 0.69, indicating an inverse exponential function relationship between noise value and exposed aggregate depth. This relationship between noise value and exposed aggregate depth also reflects the variation pattern of the air pumping effect

generated by tires traveling at a certain speed on EAC pavement. Specifically, as the exposed aggregate depth increases, the voids between the tire and the EAC surface during contact increase, and the air discharge channels become more unobstructed, reducing the likelihood of forming air compression in enclosed voids. Consequently, the air pumping effect between the tire and the pavement weakens, and the surface noise value of the pavement gradually decreases.

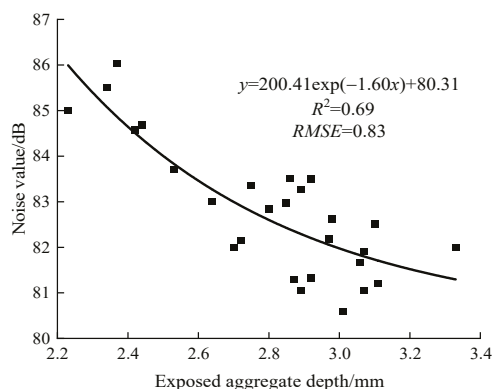


Figure 12. The relationship between noise value and exposed aggregate depth.

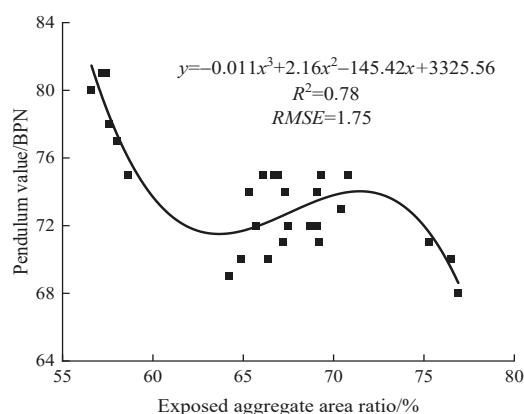


Figure 13. The relationship between pendulum value and exposed aggregate area ratio.

As can be seen from Figure 13, as the exposed aggregate area ratio increases, the pendulum value shows a trend of first decreasing, then increasing, and finally decreasing again, forming a cubic polynomial relationship with a coefficient of determination R^2 reaching 0.78. This variation trend between the pendulum value and exposed aggregate area ratio mainly depends on the contact condition between the rubber slider of the pendulum friction tester and the surface texture of EAC during testing. Generally, the mortar texture contains a large number of rich micro-textures, and under the condition that the rubber slider simulates the low-speed driving of automobile tires, the micro-textures play a major role in pavement skid resistance. Therefore, when the exposed aggregate area ratio is less than 64%, the larger the exposed aggregate area ratio, the smaller the contact area between the rubber slider and the mortar texture, and the lower the skid resistance performance of the EAC surface. When the exposed aggregate area ratio is between 64% and 71%, the coarse aggregate texture becomes the main texture. At this time, an increase in the exposed aggregate area ratio reduces the contact between the rubber slider and part of the mortar texture, reducing the impact of the mortar texture on skid resistance performance, and the friction effect of the coarse aggregate on the rubber slider becomes the main source of skid resistance performance. Therefore, as the exposed aggregate area ratio increases, the pendulum value gradually increases. When the exposed aggregate area ratio

exceeds 71%, the mortar texture distributed in the valleys between the coarse aggregates is almost completely blocked by the coarse aggregate texture, and the texture contacted by the rubber slider changes from the original composite texture of coarse aggregate and mortar to only contacting the coarse aggregate texture. Therefore, the pendulum value shows a downward trend once again.

In summary, the SRNR performance of EAC is closely related to its surface texture characteristics. However, some data points in the fitting results exhibited relatively high dispersion, with root mean square errors (RMSEs) of 1.75 BPN for the pendulum value and 0.83 dB for the noise value. This indicates that while the exposed aggregate depth and area ratio can reflect the overall variation trends of SRNR performance for EAC, high-precision prediction remains challenging due to the influence of additional factors such as the exposure postures of coarse aggregates and surface microtexture. Based on the correlation analysis, under the condition of meeting the basic requirements for surface texture characteristics with an exposed aggregate depth of 2~3 mm and an exposed aggregate area ratio of 65~75%, the greater the exposed aggregate depth, the better the noise reduction performance of the EAC surface. When the exposed aggregate area ratio is controlled between 69% and 73%, the skid resistance performance of its surface reaches a high and stable level.

5. Conclusions

To enhance the SRNR performance of EAC and clarify the formation mechanism of its surface texture characteristics and their relationship with SRNR performance, this paper investigated the effects of four mix design parameters—water–binder ratio, sand ratio, coarse aggregate volume ratio, and proportion of aggregates > 9.5 mm—on the surface texture characteristics, SRNR performance, and mechanical properties of EAC through orthogonal experiments. The optimal mix proportions for EAC were proposed, and a correlation analysis was conducted between the surface texture characteristics and the SRNR performance. The main conclusions are as follows:

- (1) The proportion of aggregates > 9.5 mm is the most significant factor affecting the surface texture characteristics and SRNR performance of EAC. When the proportion of aggregates > 9.5 mm is 70%, both the exposed aggregate depth and the exposed aggregate area ratio of EAC decrease significantly. Therefore, the proportion of aggregates > 9.5 mm should not be too high during mix design.
- (2) The optimal mix proportions of EAC are as follows: water–binder ratio of 0.43, sand ratio of 31%, coarse aggregate volume ratio of 42%, and proportion of aggregates > 9.5 mm of 50%. Compared with ordinary grooved concrete, the optimal mix proportions of EAC increase the pendulum value by 31.5% and reduce the noise value by 6.48 dB. The frequency range of noise reduction is mainly concentrated between 1.5 kHz and 4.0 kHz, which belongs to the mid-high frequency range that is more sensitive to the human ear.
- (3) There is a high correlation between noise value and exposed aggregate depth, as well as between pendulum value and exposed aggregate area ratio. Specifically, as the exposed aggregate depth increases, the noise value gradually decreases and stabilizes, showing an inverse exponential function relationship. On the other hand, as the exposed aggregate area ratio increases, the pendulum value first decreases, then increases, and finally decreases again, exhibiting a cubic polynomial relationship.

EAC has broad application prospects in the field of pavement engineering due to its superior SRNR performance. Future research will employ 3D laser scanning technology and machine learning algorithms [40–43] to establish the relationship between macroscopic/microscopic surface texture parameters and SRNR performance of EAC, in-

vestigating the evolution patterns of texture parameters and SRNR performance under different wear levels, so as to provide a reference for the practical engineering application effects of EAC.

Author Contributions: Conceptualization, C.W. and X.Z.; methodology, C.W. and R.L.; software, C.W.; validation, C.W. and R.L.; formal analysis, X.Z.; investigation, C.W.; resources, X.Z.; data curation, C.W.; writing—original draft preparation, C.W.; writing—review and editing, X.Z. and R.L.; visualization, Y.L.; supervision, X.Z. and Y.L.; project administration, X.Z. All authors have read and agreed to the published version of the manuscript.

Funding: This work was funded by National Natural Science Foundation of China [grant number 51878077] and the Postgraduate Scientific Research Innovation Project of Hunan Province [grant number CXCLY2022002].

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding author.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Fang, J.; Tu, J.; Wu, K. Analysis of skid resistance and noise characteristics for varieties of concrete pavement. *Adv. Mater. Sci. Eng.* **2020**, *1*, 7427314. [CrossRef]
2. Zhao, W.; Zhang, J.; Lai, J.; Shi, X.; Xu, Z. Skid resistance of cement concrete pavement in highway tunnel: A review. *Constr. Build. Mater.* **2023**, *406*, 133235. [CrossRef]
3. Wen, B.; Cao, D. Statistical analysis of traffic accidents and skid resistance of expressway pavement. *HTRD* **2006**, *23*, 72–75. [CrossRef]
4. Zhang, C.; Wang, B.; He, J.; Xiang, D.; Yi, L. Traffic risk analysis of ponding sections on freeways based on driving dynamics. *J. Transp. Inf. Saf.* **2019**, *37*, 9–17. [CrossRef]
5. Kassomenos, P.; Vogiatzis, K.; Coelho, J. Critical issues on environmental noise: Editorial. *Sci. Total Environ.* **2014**, *482*, 399–400. [CrossRef]
6. Miera-Dominguez, H.; Lastra-González, P.; Indacoechea-Vega, I.; Castro-Fresno, D.; Van Loon, R.; van Blokland, G.; Muresn, B.; Blanc, J. Development and validation of a new asphalt mixture to reduce tyre-road contact noise in interurban areas. *Constr. Build. Mater.* **2024**, *453*, 139025. [CrossRef]
7. Zhu, B.; Hu, D.; Liao, F.; Chen, J.; Su, B.; Wu, J.; Wang, Y. A Fast Approach to Optimize Tread Pattern Shape for Tire Noise Reduction. *Appl. Sci.* **2023**, *13*, 10256. [CrossRef]
8. Ouis, D. Annoyance from road traffic noise: A review. *J. Environ. Psychol.* **2001**, *21*, 101–120. [CrossRef]
9. Wegman, F. The future of road safety: A worldwide perspective. *IATSS Res.* **2017**, *40*, 66–71. [CrossRef]
10. Cai, Z.; He, Z.; Tian, B.; Qiu, Z. Study on effecting depth of aggregate exposing solvent. *Highw. Eng.* **2018**, *43*, 66–69+100. [CrossRef]
11. Cai, Z.; He, Z.; Wang, Z.; Yu, D.; Hou, Z.; Zhang, Y.; Zhang, B. Study on the amount of cleaning of exposed-aggregate concrete pavement. *Concrete* **2018**, *3*, 149–152. [CrossRef]
12. Wei, L. Study on surface characteristics of exposed aggregate concrete pavement. *Low Temp. Archit. Technol.* **2016**, *38*, 17–19. [CrossRef]
13. Kim, Y.K.; Kim, J.H.; Rith, M.; Lee, S.W. Optimisation of mixture design and surface texture for low-noise exposed aggregate concrete pavements. *Int. J. Pavement Eng.* **2020**, *23*, 950–956. [CrossRef]
14. Yan, Q.; Li, X.; Tian, B.; Hou, Z.; Xie, J. Research on mix proportion design of low noise exposed-aggregate cement concrete. *Constr. Technol.* **2015**, *44*, 63–66. [CrossRef]
15. Chhay, L.; Kim, J.; Lee, S.W. Mean texture depth estimation of exposed aggregate concrete pavement surface texture based on photogrammetry technique. *Int. J. Civ. Eng.* **2024**, *22*, 1717–1729. [CrossRef]
16. Yang, L.; Chen, J.; Zhang, B.; Li, J.; Wang, F. Research on long-term anti-sliding performance of exposed aggregate cement concrete. *J. Traffic Eng.* **2020**, *20*, 1–6. [CrossRef]
17. Chen, W.; Shen, P. Research on development and pavement performances of aggregate exposing concrete. *Highway* **2013**, *58*, 45–52. [CrossRef]

18. Hu, L.; Yun, D.; Gao, J.; Tang, C. Monitoring and optimizing the surface roughness of high friction exposed aggregate cement concrete in exposure process. *Constr. Build. Mater.* **2020**, *230*, 117005. [CrossRef]
19. Rith, M.; Kim, Y.K.; Lee, S.W. Characterization of long-term skid resistance in exposed aggregate concrete pavement. *Constr. Build. Mater.* **2020**, *256*, 119423. [CrossRef]
20. Han, S.; Dong, Y.; Chen, H.; Zhang, D.; Lu, X.; Shi, Y. Noise reduction performance of exposed-aggregate cement concrete pavement. *J. Traffic Transp. Eng.* **2005**, *5*, 32–34. [CrossRef]
21. Zhao, D.; Zhang, P.; Tian, B.; Huang, Z.; Li, L. Study on the wear behavior of three-dimensional morphology of anti-sliding structure of cement concrete pavement. *New Build. Mater.* **2023**, *50*, 117–122. [CrossRef]
22. Hablovicova, B.; Krivanek, V.; Markova, P. Comparison of exposed aggregate cement concrete surface and stone mastic asphalt surface noise emissions by close-proximity method. *Appl. Sci.* **2021**, *11*, 10359. [CrossRef]
23. *JTG/T F30-2014*; Technical Guidelines for Construction of Highway Cement Concrete Pavements. Ministry of Transportation of PRC: Beijing, China, 2014.
24. Liu, Y.; Liu, M.; Qu, L.; Wang, F. Factor analysis affect the surface structure of exposed-aggregate cement concrete pavement. *Concrete* **2016**, *2*, 157–160. [CrossRef]
25. Šernas, O.; Zofka, A.; Vaitkus, A.; Gražulytė, J. The effect of exposed aggregate concrete gradation on the texture characteristics and durability. *Constr. Build. Mater.* **2020**, *261*, 119921. [CrossRef]
26. Hu, S. Study on mix proportion of slipform paving of tunnel exposed aggregate cement concrete pavement. *Commun. Sci. Technol. Heilongjiang* **2022**, *45*, 113–117. [CrossRef]
27. Xiao, Y. Application of exposed aggregate concrete pavement in road tunnel construction. *Transp. World* **2023**, *36*, 154–156. [CrossRef]
28. Jang, G. Analysis of construction techniques for high skid resistance and low noise exposed aggregate cement concrete pavement. *Transp. World* **2024**, *17*, 65–67. [CrossRef]
29. *JGJ 55-2011*; Specification for Mix Proportion Design of Ordinary Concrete. Ministry of Transportation of PRC: Beijing, China, 2011.
30. Yin, F.; Cheng, X. Key construction control technology of exposed aggregate cement concrete. *Highw. Transp. Inn. Mong.* **2007**, *6*, 15–16. [CrossRef]
31. Zha, X.; Liu, L.; Bai, L.; Han, X. Image recognition method of asphalt-aggregate ratio for asphalt mixture. *J. Traffic Transp. Eng.* **2013**, *13*, 1–6+14. [CrossRef]
32. *JTG 3450-2019*; Field Test Methods of Highway Subgrade and Pavement. Ministry of Transportation of PRC: Beijing, China, 2019.
33. Mai, J.; Meng, X.; Li, L.; Zhao, L.; He, J. Study on denoising and sound absorption performance of drainage asphalt pavement based on indoor tire accelerated falling method. *J. Highw. Transp. Res. Dev.* **2022**, *39*, 17–24. [CrossRef]
34. *JTG 3420-2020*; Testing Methods of Cement and Concrete for Highway Engineering. Ministry of Transportation of PRC: Beijing, China, 2020.
35. Panagoulis, O.K.; Kokkalis, A.G. Skid resistance and fractal structure of pavement surface. *Chaos Soliton. Fract.* **1998**, *9*, 493–505. [CrossRef]
36. Han, D.; Wang, L.; Guo, P.; Lu, W.; Yang, X. Review of principles and research on noise reduction technology for dense-graded rubber asphalt pavements. *J. Chang. Univ. Sci. Technol. (Nat. Sci. Ed.)* **2024**, *21*, 61–78+106. [CrossRef]
37. Huang, Z.; Zang, X.; Chen, B.; Yu, J.; Chen, F. Influence of tyre contact configuration distribution on noise reduction characteristics of asphalt pavement. *Sci. Technol. Eng.* **2018**, *18*, 123–128. [CrossRef]
38. *JTG D40-2011*; Specifications for Design of Highway Cement Concrete Pavement. Ministry of Transportation of PRC: Beijing, China, 2011.
39. Guo, C.; Shen, G.; Wang, X.; Li, L. Analysis of asphalt-rubber pavement noise spectrum with close-proximity trailer method. *China J. Highw. Transp.* **2012**, *25*, 22–28. [CrossRef]
40. Dong, S.; Han, S.; Xu, J.; Su, H.; Niu, D.; Chen, D.; Jia, M.; Wang, W. Review of methods for three-dimensional reconstruction and evaluation of asphalt pavement textures. *China J. Highw. Transp.* **2025**, *38*, 60–84. [CrossRef]
41. Asteris, P.G.; Skentou, A.D.; Bardhan, A.; Samui, P.; Pilakoutas, K. Predicting concrete compressive strength using hybrid ensembling of surrogate machine learning models. *Cement Concrete Res.* **2021**, *145*, 106449. [CrossRef]
42. Jamhiri, B.; Jalal, F.E.; Chen, Y. Hybridizing multivariate robust regression analyses with growth forecast in evaluation of shear strength of zeolite-alkali activated sands. *Multiscale Multi. Mod.* **2022**, *5*, 317–335. [CrossRef]
43. Naser, A.H.; Badr, A.H.; Henedy, S.N.; Ostrowski, K.A.; Imran, H. Application of multivariate adaptive regression splines (mars) approach in prediction of compressive strength of eco-friendly concrete. *Case Stud. Constr. Mat.* **2022**, *17*, e01262. [CrossRef]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

A Study on the Creep Characteristics of Gassy Clay Mixed with Silt

Aiwu Yang, Tianli Liu, Hao Zhang * and Boqu Zhang

Department of Civil Engineering, College of Environmental Science and Engineering, Donghua University, Shanghai 201620, China

* Correspondence: hzhang@dhu.edu.cn

Abstract: As the economy evolves, there has been an increasing interest in exploring oceanic resources. However, the complex marine environment poses several geological challenges for offshore engineering endeavors. The presence of gassy soil significantly influences the deformation properties and integrity of the soil, significantly impacting offshore engineering construction. Triaxial shear tests and creep tests were conducted on gassy clay with silt content, prepared using the laboratory “zeolite method”, to analyze its shear deformation characteristics and long-term resilience. We proposed a prediction model for calculating the long-term resilience of silt-containing clay, accounting for confining pressure and gas content, and verified its efficacy through experimentation. Our findings reveal the following: The stress–strain relationship curve of silt-containing gassy clay is a typical strain hardening curve. The greater the confining pressure or the smaller the gas content, the greater the stress under the same strain and the greater the yield stress; when the gas content is the same, the greater the confining pressure, the greater the long-term strength of the soil; and when the confining pressure is the same, the smaller the gas content, the greater the long-term strength of the soil. The research results can provide theoretical reference for actual complex engineering.

Keywords: gassy clay with silt content; creep characteristics; long-term resilience; gas content; creep model

1. Introduction

Gassy soil primarily contains CH_4 , N_2 , CO_2 , etc., collectively referred to as shallow gas. Shallow gas represents a natural gas resource and a geological hazard, especially in oceanic environments. In China, shallow gas is widely dispersed along the southeastern coast, the middle and lower reaches of the Yangtze River, the Sichuan Basin, and the Bohai Bay Basin, with an even broader presence on the seabed. Its impact on the construction of foundation engineering projects is often considerable [1–3].

Scholars both domestically and internationally have conducted extensive studies on the mechanical properties of gassy sands and gassy soils: Tong et al. [4] conducted triaxial tests on saturated sand and gas-bearing sand and established a constitutive model of gas-bearing sand and a static liquefaction criterion for gas-bearing sand. Kong et al. [5] conducted triaxial tests on Hangzhou Bay gas storage sand under four stress paths of active and passive compression and elongation and demonstrated the influence of stress path on the deformation and strength characteristics of unsaturated sand. Guo et al. [6] conducted field tests and indoor tests to compare the bearing capacity of sand before and after biogas release and found that the slow release of biogas can improve the bearing capacity of

sand. Vega-Posada et al. [7] conducted single and cyclic triaxial tests on medium-density gas-bearing sand and found that even the presence of a small amount of gas makes the soil more easily compressed and limits the accumulation of excess pore water pressure. Wang et al. [8] found that the occurrence and configuration characteristics of sand in the shallow gas-rich zone of a reservoir were similar to those of ordinary unsaturated soil, but its pore gas pressure was several times that of the atmospheric pressure. During the gas release process, the deformation of the reservoir sand caused only by a reduction in suction was negligible, and the main deformation was attributed to the increase in net average stress caused by a reduction in gas pressure in the reservoir. Yang et al. [9] studied the influence of temperature on the shear behavior of gas-bearing sand and found that, under undrained shear conditions, the shear strength of loose gas-bearing sand increased with increasing temperature, while the opposite was true for dense gas-bearing sand, and the higher the gas content, the more obvious the phenomenon.

The aforementioned research on the mechanical properties of gassy sands is relatively complete. However, due to the differences in particle size, structure and mineral composition, the mechanical properties of gassy soils are also different to those of gassy sands: Song et al. [10] conducted a K_0 consolidation test on gas-bearing soil prepared by zeolite and found that the compression coefficient of gas-bearing soil was negatively correlated with the load, and the compression coefficient and rebound coefficient were both positively correlated with the load and gas content. Ye et al. [11] studied the pore pressure dissipation and settlement development law of fine-grained gas-bearing soil and found that the instantaneous settlement was negatively correlated with saturation, the instantaneous settlement was positively correlated with pore pressure, and the final settlement of gas-bearing soil was greater than that of saturated soil. Wang et al. [12] conducted indoor tests to study the distribution of water vapor pressure and saturation changes in the aeration zone of shallow gas reservoirs and their effects on soil deformation under controlled degassing measures. Xu et al. [13] conducted undrained triaxial compression tests on gas-bearing soil and found that the presence of bubbles increased the undrained shear strength of loose sand but reduced the shear strength of dense sand. Wang et al. [14] prepared gas-bearing clay using the zeolite method and conducted triaxial and one-dimensional consolidation tests. They found that the trapped gas in the soil hindered drainage, increased the undrained shear strength of the soil and reduced the consolidation coefficient. Hong et al. [15] conducted undrained cyclic triaxial tests on normally consolidated fine-grained gas-bearing soil and found that during the cyclic shear process, the collapse of gas-filled pores led to an increase in cyclic pore water pressure and a decrease in stiffness, while water partially discharged into the gas cavity could prevent this process. Liu et al. [16] compared the drained stiffness and undrained stiffness of gas-containing and non-gas-containing fine-grained sediments and found that stress history had almost no effect on the undrained stiffness of gas-containing fine-grained sediments, while the drained stiffness of non-gas-containing fine-grained sediments was related to stress history. Sultan et al. [17] conducted triaxial tests on natural gas-containing sediments in the Gulf of Guinea. The results showed that the expansion and dissolution of gas led to an increase in the compression rate of the soil, a decrease in the preconsolidation pressure and the undrained shear peak strength, and an increase in the pore pressure. Wang et al. [18] analyzed the effect of water content on the strength and deformation characteristics of thin silt sand interlayers in silty gas-containing clay in a tidal flat area in Cangnan through triaxial tests and consolidation tests. The results showed that the soil cohesion was negatively correlated with the water content, while the internal friction angle was almost unaffected by the water content. Hyodo et al. [19] compared the triaxial compression properties of gas-saturated soil and water-saturated soil and found that the stiffness and strength of gas-saturated samples were higher than those

of water-saturated samples, and the strain softening behavior of gas-saturated samples was more obvious and the volumetric strain was larger. Yoneda et al. [20] used a mixed pressure coring system to recover cores from shallow natural gas hydrate deposits on the deep seabed and conducted uniaxial and triaxial compression tests to obtain strength and stiffness data for shallow natural gas hydrate deposits. The abovementioned research on gassy soils is relatively advanced but mostly consists of simple experimental studies such as triaxial tests or cross-plate shear tests. In particular, for gassy soft clay, due to its soft characteristics after sample preparation, existing studies rarely involve it.

This paper, based on the actual “Cross-River Tunnel” project and considering practical unconsolidated and undrained engineering conditions, conducted triaxial shear tests and creep tests on gassy soft clay. It analyzed the shear deformation characteristics and long-term strength properties of gassy clay containing silt. A long-term strength prediction model for silty clay considering confining pressure and gas content is proposed, and the effectiveness of the prediction model is verified through experiments.

2. Materials and Methods

2.1. Properties of the Natural Soil

The natural soil utilized in this study is sourced from a silt-containing clay layer located between 8.1 and 29.9 m within borehole DZT02, situated along the north bank of the river. Characterized by a gray-black hue and a distinct odor of decay, this soil exhibits significant plasticity. Table 1 presents the primary physical attributes of the natural soil, while Figure 1 depicts the results of the particle analysis.

Table 1. Basic physical properties of silt-containing clay.

Water Content	Density	Specific Gravity	Porosity Ratio	Liquid Limit	Plastic Limit
ω	ρ	G_s	e	ω_l	ω_p
38.83%	1.78 g/cm ³	2.72	1.126%	37.32%	25.10%

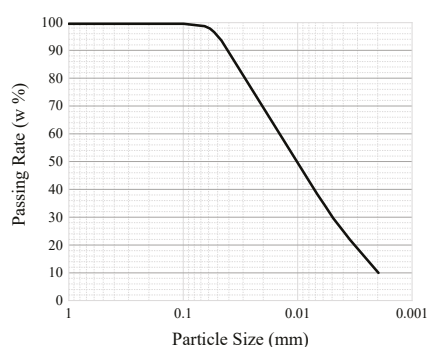


Figure 1. Particle size distribution curve of clay sample.

2.2. Preparation of Soil with Different Gas Content

This study employs the “zeolite method” to simulate the preparation of modified samples of gassy clay with added silt. Zeolite powder serves as a medium for introducing gas into the gassy soil, with N₂ chosen as the experimental gas. The natural soil undergoes a series of steps including drying, grinding, and sieving to eliminate large particles and impurities. Subsequently, the dried zeolite powder is placed in a pressure chamber and subjected to a vacuum for 2 h, reducing the pressure within the chamber to −100 kPa, while the zeolite powder remains in this environment for 24 h to ensure complete gas removal from the zeolite crystals. A nitrogen gas pressurization device is then connected to the pressure chamber, and over a span of 24 h, nitrogen gas at 200 kPa is intermittently injected

into the chamber every 6 h to saturate the zeolite crystals with nitrogen gas. The nitrogen-filled zeolite powder is carefully mixed with dry soil particles no larger than 0.075 mm, following the proportions detailed in Table 2. Distilled water is added to the mixture, which is then thoroughly stirred. The transfer of the soil sample into an acrylic molding tube for self-weight consolidation is conducted swiftly, with the stirring-to-sampling process controlled to within 2 min to minimize nitrogen gas loss. The molding tube is sealed and left to stand for 24 h, facilitating the complete displacement of gas within the zeolite crystals by water and enabling the gassy soil to consolidate under its own weight to a specific strength capable of supporting the pre-consolidation pressure.

Table 2. Soil sample component ratio.

Zeolite Content (%)	Gas Content (%)	Dry Soil Weight (g)	Zeolite Weight (g)	Distilled Water Weight (g)	Total Weight (g)	Water Content (%)
0	0	500	0	500	1000	100
2	8.36	490	10			
5	14.07	475	25			
10	25.24	450	50			

2.3. Triaxial Shear Test Study

The soil sample undergoes pre-consolidation within a dedicated device, where vertical stress is incrementally applied, starting from 3 kPa and increasing by 3 kPa every 6 h until reaching 24 kPa, marking the completion of pressurization. Following pre-consolidation, the soil sample's resilience is further increased, paving the way for conventional triaxial shear tests conducted utilizing an advanced stress path triaxial rheometer. In alignment with the actual conditions of the engineering site, test conditions are established as unconsolidated and undrained, with confining pressures set at 100 kPa, 150 kPa, 200 kPa, and 250 kPa. The testing procedure adheres to the specifications outlined in the “Chinese Standard for Geotechnical Test Methods (GB/T50123-2019)”.

2.4. Creep Test Study

The creep test comprises two parts, examining the creep characteristics of silt-containing gassy clay with identical gas content and different confining pressures and with identical confining pressure and varying gas contents. Utilizing a triaxial apparatus, the experimental setup features specimen dimensions of $\Phi 39.1 \text{ mm} \times \text{H}80 \text{ mm}$. Employing the step loading method, the test utilizes the deviatoric stress amplitude from the conventional triaxial shear test (as detailed in Section 2.3) as the maximum deviatoric stress in the creep test. The loading time for each deviatoric stress level is set at 50 h, with the specific loading schemes outlined in Tables 3 and 4.

Table 3. Loading path of test 1 (with identical gas content and different confining pressures).

Gas Content W (%)	Confining Pressure σ_3 (kPa)	Deviatoric Stress $q = \sigma_3 - \sigma_3$ (kPa)
14.07	100	15–30–45–60
	150	25–50–75–100
	200	30–60–90–120
	250	40–80–120–160

Table 4. Loading path of test 2 (with identical confining pressure and varying gas contents).

Confining Pressure σ_3 (kPa)	Gas Content W (%)	Deviatoric Stress $q = \sigma_3 - \sigma_3$ (kPa)
150	0	15–30–45–60
	8.36	30–60–90–120
	14.07	25–50–75–100
	25.24	15–30–45–60

3. Results

3.1. Analysis of Triaxial Shear Characteristics

Following the triaxial shear tests, stress–strain relationship curves of the silt-containing gassy clay are plotted under various confining pressures with identical gas content and under consistent confining pressure with differing gas contents, depicted in Figure 2 (take the gas content of 14.07% and the confining pressure of 150 kPa as an example). Yield stress distribution curves of the silt-containing gassy clay are plotted under the dual influence of confining pressures and gas contents, depicted in Figure 3.

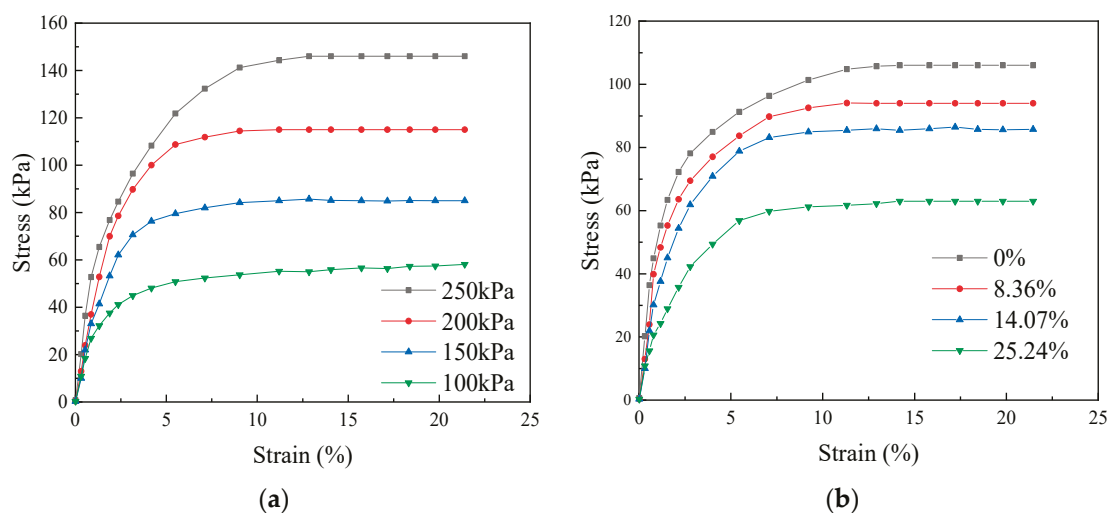


Figure 2. (a) Conventional triaxial test curves under different confining pressures (at the gas content of 14.07%); (b) conventional triaxial test curves under different gas content levels (at the confining pressure of 150 kPa).

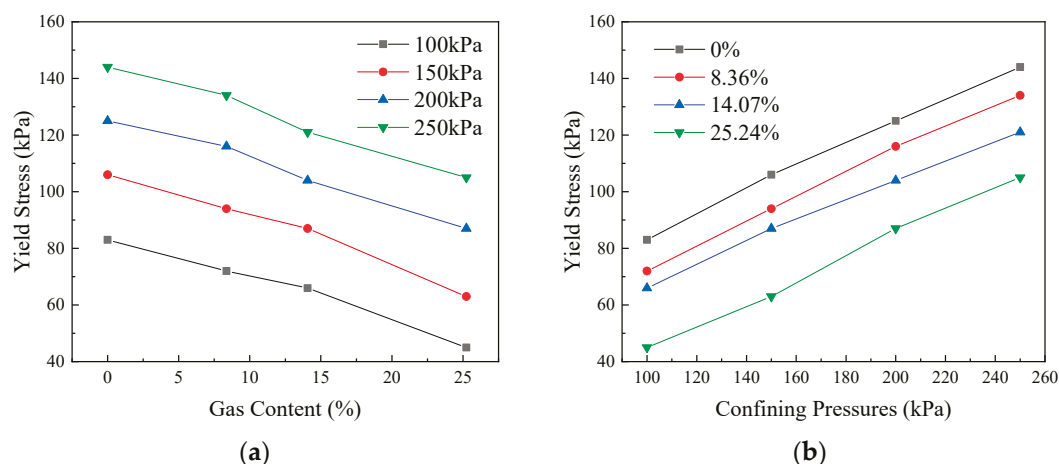


Figure 3. Yield stress distribution curves of the silt-containing gassy clay under the dual influence of confining pressures and gas contents (a) Yield stress–gas contents curves under different confining pressures; (b) Yield stress–confining pressures curves under different gas contents.

An analysis of Figure 2a reveals that under four distinct confining pressures, stress increases with strain when the specimen's strain is minimal; as strain reaches a certain threshold, stress remains almost unchanged; when the strain continues to increase, the stress eventually reaches a stable state and no peak value appears, indicating that the stress–strain relationship curve of silt-containing gassy clay exhibits a characteristic strain-hardening pattern [21]. Additionally, Figure 2a underscores the significant influence of confining pressure on the stress–strain characteristics of the specimen when the gas content remains constant. Higher confining pressures correspond to greater stress levels exhibited by the specimen at identical strain levels, alongside elevated yield stress and the corresponding strain.

From Figure 2b, it is evident that for the four different gas contents, as the strain on the specimen remains minimal, the stress increases proportionally with strain; as the strain reaches a certain threshold, the stress relatively stabilizes; and when the strain continues to increase, the stress eventually reaches a constant state and no peak value appears, indicating that the stress–strain curve of silt-containing gassy clay demonstrates a typical strain-hardening pattern [21]. Additionally, Figure 2b illustrates the significant impact of specimen gas content on its stress–strain relationship under identical confining pressures. Specifically, lower gas content correlates with higher stress levels at equivalent strains, resulting in increased yield stress and strain. Notably, when the gas content of the specimen is zero, stress peaks at the same strain, as do the yield stress and its corresponding strain.

Comparing Figure 2a,b reveals consistent trends in the stress–strain relationship curves of the specimens. Both sets exhibit strain-hardening behavior. Notably, variations in confining pressure and gas content markedly influence the stress–strain relationship. Higher confining pressure or lower gas content results in greater stress levels at an equivalent strain, alongside elevated yield stress and strain.

It can be seen from Figure 3 that at each level of confining pressure, the yield stress of the soil sample tends to decrease with the increase in gas content, and the relationship is linear; at each level of gas content, the yield stress of the soil sample tends to increase with the increase in confining pressure, and the relationship is also linear. According to the phenomenon in Figure 3, we believe that the gas content directly affects the yield stress but has no effect on the relationship between the yield stress and the confining pressure. Similarly, the confining pressure directly affects the yield stress but has no effect on the relationship between the yield stress and the gas content; that is,

$$\frac{\partial^2 \tau}{\partial W \partial \sigma_3} = \frac{\partial^2 \tau}{\partial \sigma_3 \partial W} = 0. \quad (1)$$

3.2. Analysis of Creep Characteristics

Based on the test loading schemes outlined in Tables 3 and 4, triaxial creep tests were conducted on the prepared silt-containing gassy clay. Subsequent to the creep tests, full-process creep curves were plotted for the silt-containing gassy clay under identical gas content with varying confining pressures and under identical confining pressure with varying gas contents, as illustrated in Figures 4 and 5.

Figures 4 and 5 reveal that with consistent gas content and varying confining pressures, or with consistent confining pressure and varying gas contents, the axial strain of the specimen exhibits a consistent pattern of change over time following the application of each deviatoric stress level. The axial strain gradually increases with time, ultimately reaching stabilization, indicating a decaying development of creep. Subsequent applications of deviatoric stress consistently result in greater axial strain compared to the preceding levels. Figure 4 further demonstrates that when the applied deviatoric stress is below the yield stress observed in conventional triaxial shear tests with the same confining pressure, the

axial strain of the specimen increases relatively slowly over time, with a smaller magnitude of increase and a smaller stable-state axial strain. Conversely, when the applied deviatoric stress approaches the yield stress observed in conventional triaxial shear tests under the same confining pressure, the increase in axial strain becomes more pronounced over time, with a larger magnitude of increase and a larger stable-state axial strain.

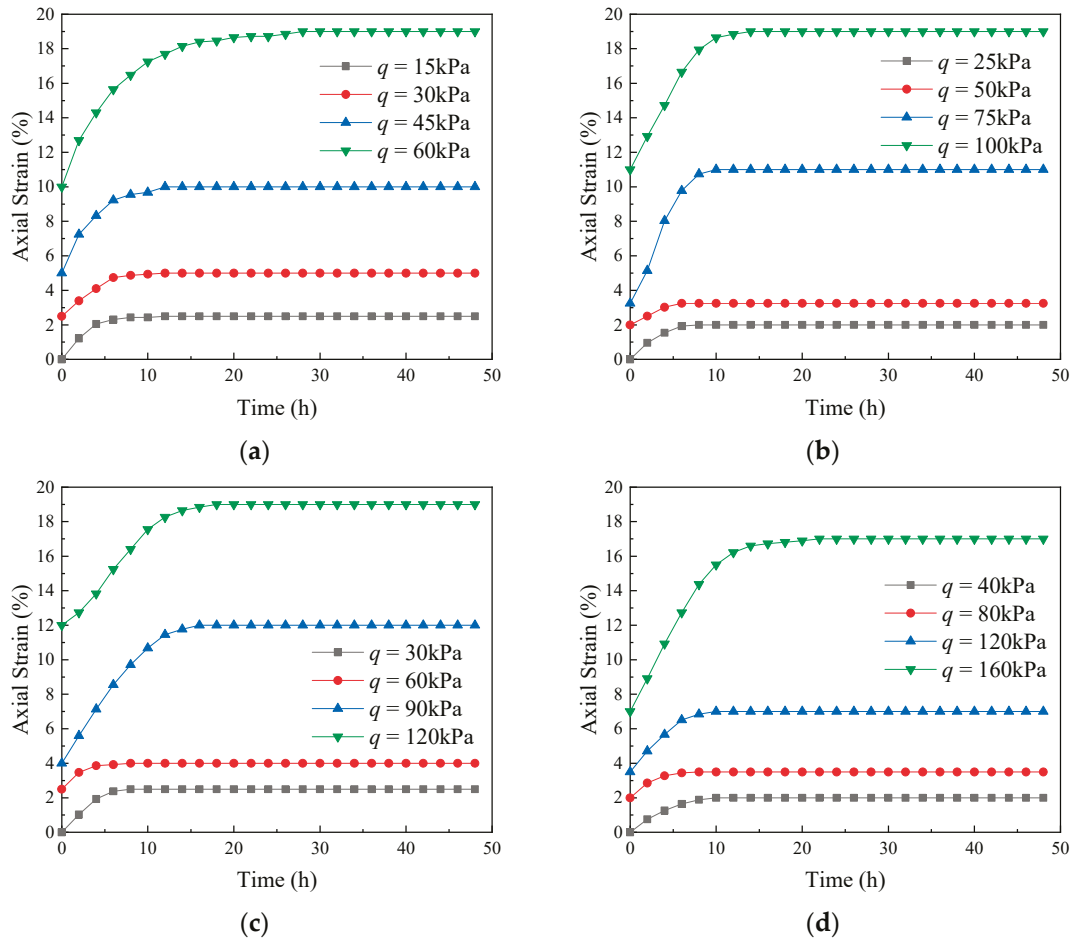


Figure 4. Full-process creep curves under different confining pressures when $W = 14.07\%$ (a) $\sigma_3 = 100$ kPa; (b) $\sigma_3 = 150$ kPa; (c) $\sigma_3 = 200$ kPa; (d) $\sigma_3 = 250$ kPa.

Analyzing the phenomena depicted in Figures 4 and 5, when the applied deviatoric stress is below the yield stress observed in conventional triaxial shear tests under the same confining pressure, the structure of the gassy soil remains intact. Consequently, the shear deformation exhibited by the specimen primarily entails elastic deformation. Conversely, when the applied deviatoric stress approaches the yield stress resulting from the same confining pressure as in conventional triaxial shear tests, the structure of the gassy soil experiences partial disruption. As a result, the shear deformation undergone by the specimen includes both elastic and plastic deformation. However, owing to the presence of confining pressure, the axial strain of the specimen can stabilize. Upon surpassing the yield stress observed in conventional triaxial shear tests with the same confining pressure, the structure of the gassy soil undergoes complete breakdown. At this juncture, the shear deformation displayed by the specimen primarily involves plastic deformation. Nevertheless, as the test's confining pressure remains at a level higher than the deviatoric stress, the specimen continues to exhibit a state of decaying creep by the end of the test.

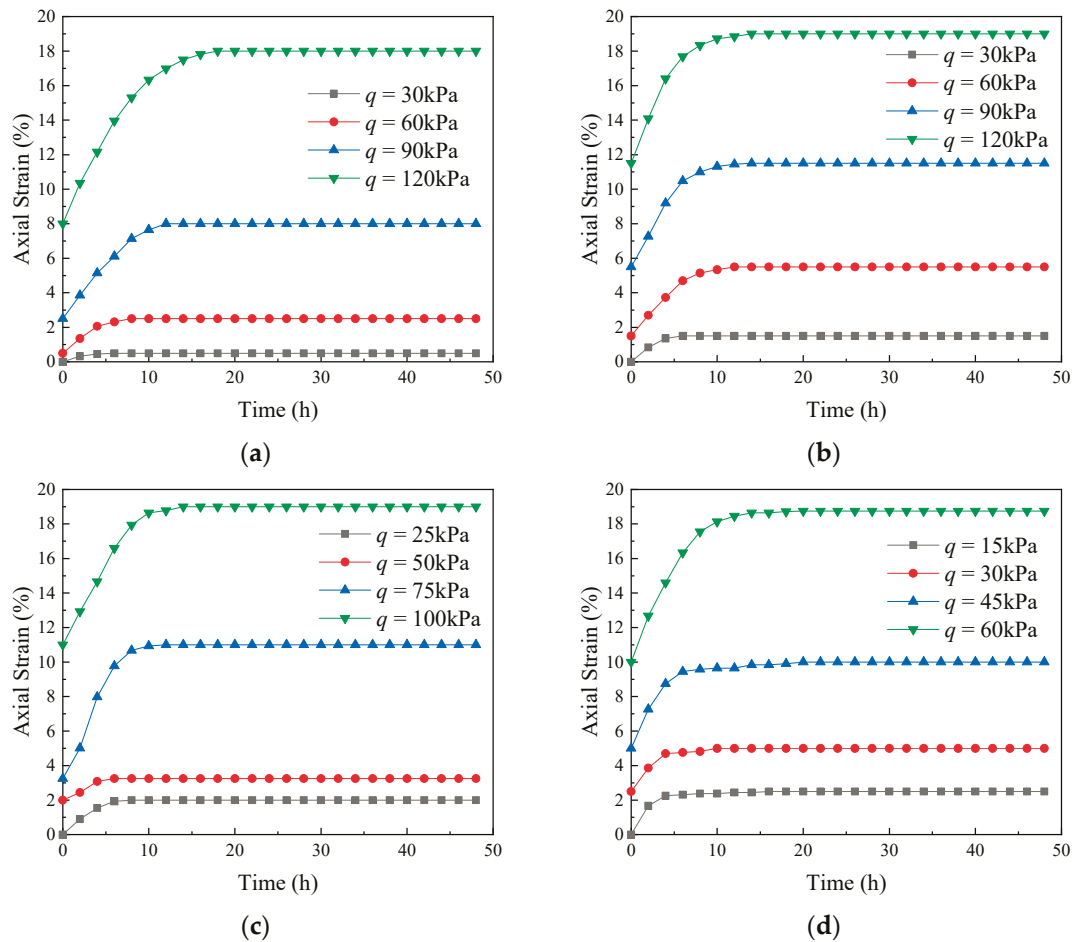


Figure 5. Full-process creep curves under different gas contents when $\sigma_3 = 150$ kPa (a) $W = 0\%$; (b) $W = 8.36\%$; (c) $W = 14.07\%$; (d) $W = 25.24\%$.

3.3. Long-Term Resilience Prediction Model

Figures 4 and 5 serve as the basis for plotting isochronous stress–strain curves during the creep process of the specimen. Time points are selected at 0, 0.5, 1, 2, 3, 4, 8, 12, 24, and 48 h, as depicted in Figures 6 and 7.

From Figures 6 and 7, it is evident that the stress–strain isochronous curves of the specimen can be roughly categorized into three stages: the first stage exhibits approximate linearity with a consistent slope, signifying the elastic stage of the specimen; the second stage demonstrates a small degree of non-linearity with a progressively decreasing slope, indicating the viscoelastic stage; and the third stage showcases significant non-linearity with a notably reduced slope, suggesting the onset of internal soil structure breakdown and the specimen's entry into the viscoplastic stage. According to the definition of long-term resilience, the deviatoric stress value corresponding to the inflection point between the second and third stages of the curve is considered the long-term resilience of the specimen.

Using the stress–strain isochronous curves in Figures 6 and 7, the long-term strength of silt-containing gassy clay was determined for similar gas content but different confining pressures and for the same confining pressure but different gas contents. The results are presented in Tables 5 and 6.

From Tables 5 and 6, we observe a positive correlation between the long-term strength of the soil and the confining pressure when the gas content remains constant. This correlation arises due to the confining pressure's role in restraining the lateral deformation of the soil body, thus maintaining its internal structure stability and augmenting the sample strength. Conversely, with a constant confining pressure, there exists a negative correlation

between the long-term resilience of the soil and gas content. This negative correlation stems from increasing porosity between soil particles of the gas in the soil, which diminishes soil integrity.

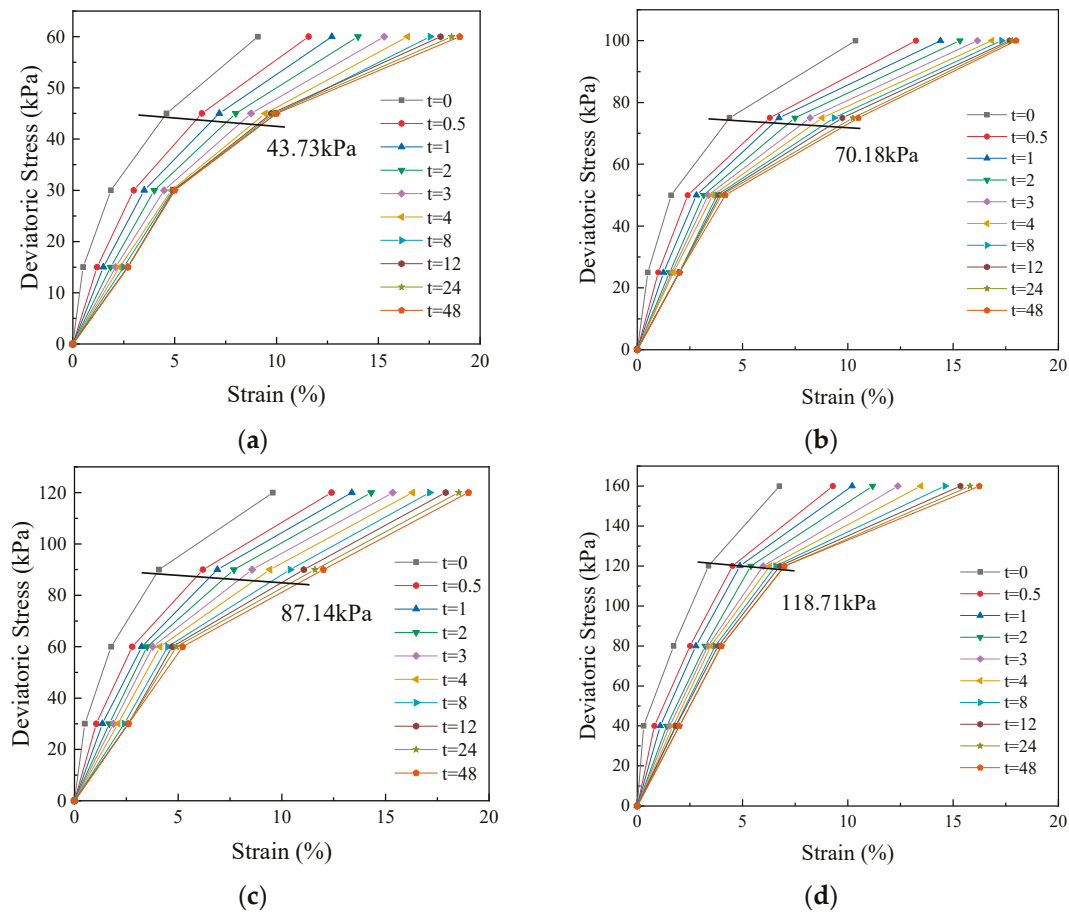


Figure 6. The isochronous curves under different confining pressures when $W = 14.07\%$ (a) $\sigma_3 = 100$ kPa; (b) $\sigma_3 = 150$ kPa; (c) $\sigma_3 = 200$ kPa; (d) $\sigma_3 = 250$ kPa.

Table 5. Long-term strength of specimens with different confining pressures when $W = 14.07\%$.

Confining Pressure (kPa)	Long-Term Strength (kPa)
100	43.73
150	70.18
200	87.14
250	118.71

Table 6. Long-term strength of specimens with different gas contents when $\sigma_3 = 150$ kPa.

Gas Content (%)	Long-Term Strength (kPa)
0	84.92
8.36	73.28
14.07	70.18
25.24	43.47

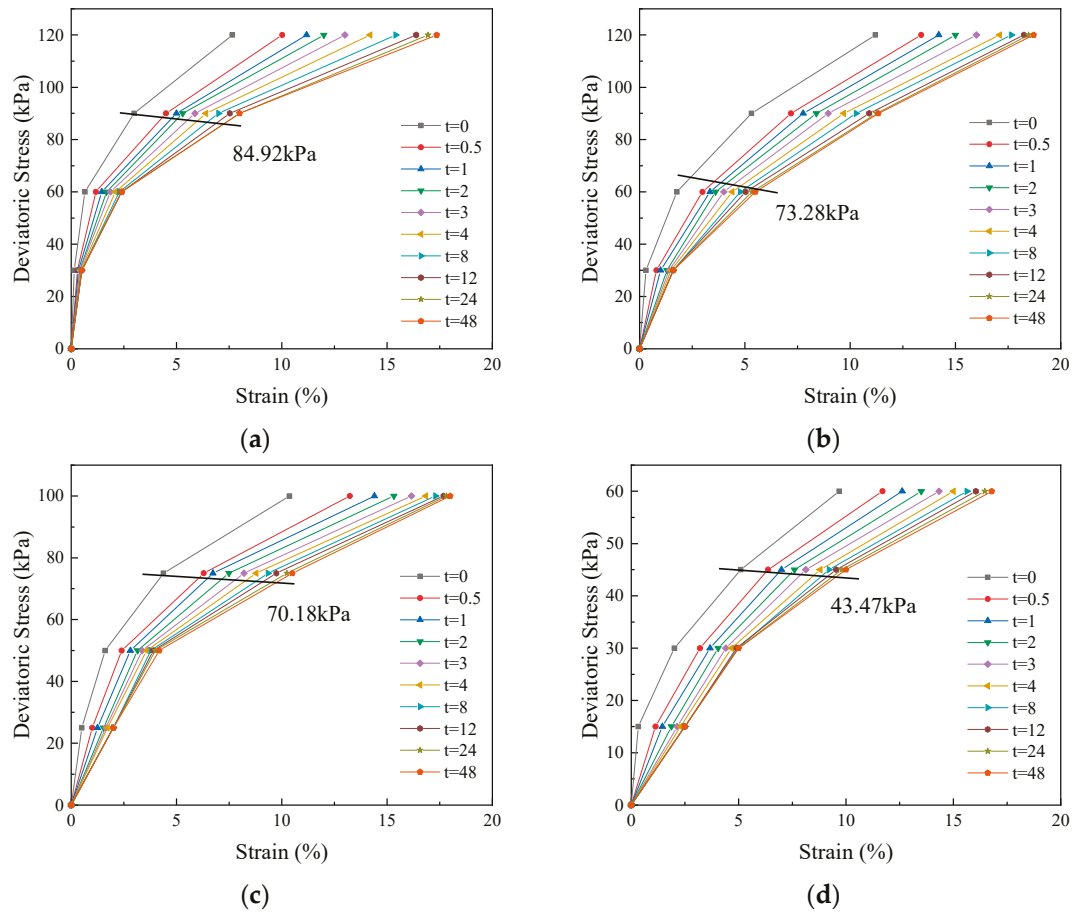


Figure 7. The isochronous curves under different gas contents when $\sigma_3 = 150$ kPa (a) $W = 0\%$; (b) $W = 8.36\%$; (c) $W = 14.07\%$; (d) $W = 25.24\%$.

Curves depicting the long-term strength of gassy soil in relation to both confining pressure and gas content are derived from Tables 5 and 6, as illustrated in Figure 8.

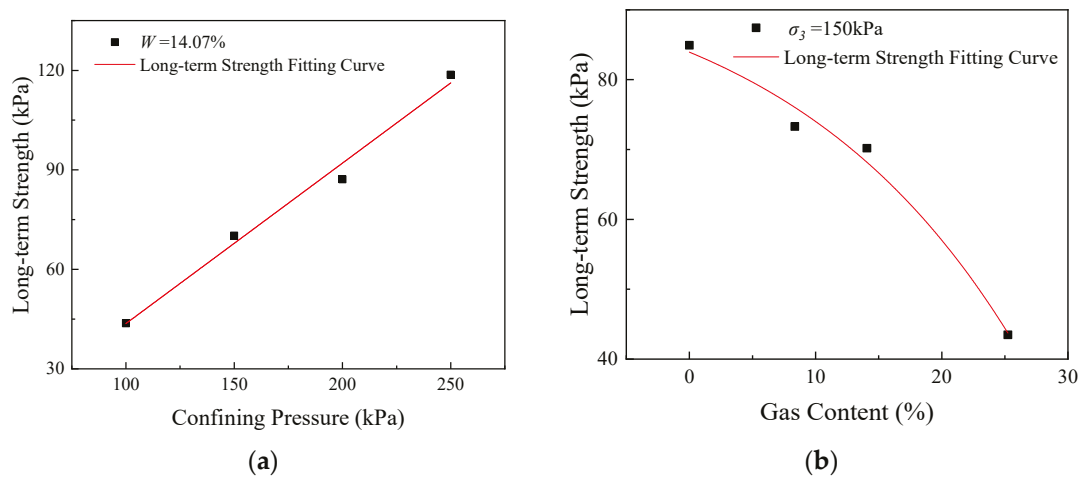


Figure 8. Relationship between long-term strength of gassy soil, confining pressure, and gas content (a) Relationship between long-term strength of gassy soil and confining pressure; (b) Relationship between long-term strength of gassy soil and gas content.

Based on Figure 8, the fitted relationship between the long-term resilience, τ , of gassy soil and the confining pressure, σ_3 , is expressed as

$$\tau = a + b \times \sigma_3, \quad (2)$$

Based on Figure 8, the fitted relationship between the long-term strength, τ , of gassy soil and the gas content, W , is described as

$$\tau = xe^{\frac{W}{y}} + z, \quad (3)$$

Integrating gas content W into both sides of Equation (1) results in the following:

$$\frac{\partial \tau}{\partial \sigma_3} = g_1(\sigma_3), \quad (4)$$

where $g_1(\sigma_3)$ can be any function with independent variable confining pressure σ_3 .

Integrating confining pressure σ_3 into both sides of Equation (4) results in

$$\tau = \int g_1(\sigma_3) d\sigma_3 + h(W), \quad (5)$$

where $h(W)$ can be any function with independent variable gas content W .

Let

$$g(\sigma_3) = \int g_1(\sigma_3) d\sigma_3, \quad (6)$$

This results in

$$\tau = g(\sigma_3) + h(W), \quad (7)$$

Substituting Equations (2) and (3) into Equation (7) results in

$$\tau = a + b \times \sigma_3 + xe^{\frac{W}{y}} + z, \quad (8)$$

Substituting the fitting results of Figure 8 into Equation (8) yields

$$\tau = -13.94e^{\frac{W}{18.59}} + 0.48 \times \sigma_3 + 25.30, \quad (9)$$

Equation (9) effectively characterizes the long-term resilience of silt-containing gassy clay.

To validate the efficacy of Equation (9), gassy soil with a gas content of 20.68% was prepared for triaxial creep tests to determine the long-term resilience of the sample. Simultaneously, the long-term resilience was calculated using Equation (9), and the results are detailed in Table 7 and Figure 9.

Table 7. Comparison between calculated and measured long-term strength results of gassy soil (at gas content of 20.68%).

Confining Pressure (kPa)	Calculated Strength (kPa)	Measured Strength (kPa)	Relative Error (%)
100	51.83	54.13	4.27
150	76.02	73.28	3.47
200	100.21	102.23	1.98
250	124.40	121.47	2.41

Table 7 and Figure 9 indicate a close proximity between the calculated and measured results of the long-term resilience of gassy soil, affirming the feasibility of employing Equation (9) for calculating the long-term resilience of silt-containing gassy clay.

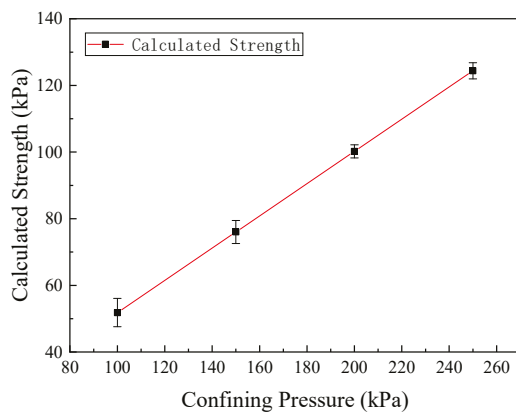


Figure 9. Error bar figure of calculated values of long-term strength of gassy clay.

To further verify whether Equation (9) can be generalized to gassy clay in other regions, we substituted the gas content and confining pressure of gassy clay (from Somerset, UK) in reference [22] into Equation (9) for calculation and compared the calculated results with the measured values in reference [22]. However, in reference [22], the addition of zeolite leads to a decrease in the saturation of gassy clay, and under different confining pressures, the rate at which the saturation of gassy clay decreases with the addition of zeolite is different. To avoid the influence of different saturations on the calculation results of Equation (9), we only selected the gas content corresponding to the same saturation (from 20% to 40%) and substituted it into Equation (9) for calculation. The results are detailed in Table 8.

Table 8. Literature data verification of Equation (9) of silt-containing gassy clay.

Test No. in Reference	Gas Content (%)	Confining Pressure (kPa)	Measured Strength in Reference (kPa)	Calculated Strength (kPa)	Relative Error (%)
21	25	200	75	67.80	9.59
22	25	200	71	67.80	4.50
26	25	200	76	67.80	10.78
33	30	400	170.5	147.30	13.61
34	30	400	172.5	147.30	14.61
37	30	400	161.5	147.30	8.79
38	30	400	169	147.30	12.84
46	25	200	76	67.80	10.78

Table 8 indicates a close proximity between the calculated and measured results of the long-term resilience of gassy soil, affirming the feasibility of generalizing Equation (9) for calculating the long-term resilience of silt-containing gassy clay in other regions.

Comparing Tables 7 and 8 reveals that when using Equation (9) to calculate silt-containing gassy clay from other regions, its relative error increases slightly but is still within an acceptable range overall.

4. Discussions

This paper uses the silty clay of a borehole 8.1–29.9 m deep in a super-large diameter cross-river tunnel project as raw soil, combines the laboratory “zeolite method” to prepare silt-containing gassy clay, and studies the mechanical properties of gassy soil such as triaxial shear strength and long-term creep strength based on triaxial shear tests and creep tests, combined with test results, theoretical analysis, and prediction models. The main research results are as follows:

(1) Indoor geotechnical tests were conducted on the raw soil to measure its basic physical and mechanical indicators. The process of preparing silt-containing gassy clay

by the “zeolite method” was introduced. At the same time, the composition ratio table of each component when preparing the soil sample and the pretreatment method of the soil sample before the triaxial test were given before the test.

(2) The triaxial test results show that the increase in confining pressure and the decrease in gas content both increase the yield stress value of silt-containing gassy clay, and the stress–strain curve relationship presents a typical strain hardening type, revealing the influence of gas content on the mechanical properties of silt-containing gassy clay, providing a basis for subsequent creep tests.

(3) From the full-process creep curve, it can be seen that the increase in gas content and confining pressure has the opposite effect on soil deformation. When other conditions are the same, the increase in confining pressure makes the deformation of silt-containing gassy clay smaller; similarly, when other conditions are the same, the increase in gas content makes the deformation of silty gas-containing clay faster. This phenomenon is due to the fact that high confining pressure can better suppress the lateral deformation of the soil and keep the soil stable, while the presence of gas in the soil changes the relatively stable state of the soil itself. The presence of gas in the soil causes the internal structure of the soil to be destroyed, and the strength of the soil is reduced, and the degree of damage increases with the increase in gas content.

(4) From the stress–strain isochronal curve, it can be seen that the increase in confining pressure has a positive effect on the structural stability of the soil, while the presence of gas weakens the structural strength of the soil itself, causing the structural attenuation of the soil under low deviatoric stress loading, which accelerates the deformation of the soil. In addition, the long-term strength is selected according to the stress–strain isochronous curve cluster method. When the gas content is constant, the long-term strength of the soil is positively correlated with the confining pressure; when the confining pressure is constant, the long-term strength of the soil is negatively correlated with the gas content.

(5) By analyzing the test results, a predictive relationship between the conventional triaxial yield stress of silty clay and the gas content and confining pressure was established, and the prediction effect was verified to be good.

(6) Compared to the complex derivation process and numerous parameters in traditional constitutive models, our long-term strength calculation model for silt-containing gassy clay can quickly calculate the long-term strength of saturated silt-containing gassy clay with fewer parameters.

Future research can be improved in the following directions:

(1) In future research, it will be possible to consider studying the triaxial creep characteristics under drainage conditions or more complex stress conditions.

(2) This paper only studies and analyzes the influence mechanism of gas content on the triaxial creep of silty clay under macroscopic conditions. In future research, it will be possible to study the microscopic mechanism of gas content before and after structural damage during creep from a microscopic perspective.

(3) We have not yet derived the long-term strength of unsaturated soil, and the influence of saturation on the long-term strength of gassy soil is also significant.

5. Conclusions

(1) The stress–strain relationship curve of gassy clay with silt content follows a typical strain-hardening pattern; confining pressure and gas content markedly affect this relationship, with higher confining pressure or lower gas content yielding higher stress at an equivalent strain, along with higher yield stress.

(2) With identical gas content, increased confining pressure enhances the long-term resilience of the soil, while lower gas content under the same confining pressure augments the soil's long-term resilience.

(3) A prediction model was proposed for calculating the long-term resilience of silty clay, taking into account both confining pressure and gas content. The efficacy of this prediction model was validated through experimentation and the literature data. The model can calculate the long-term strength of silty clay with different gas contents, has strong adaptability, and can provide theoretical reference for complex engineering practices.

Author Contributions: Conceptualization, A.Y.; writing—original draft preparation, T.L.; writing—review and editing, H.Z.; data curation, B.Z. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the National Natural Science Foundation of China, grant number 42377141; National Natural Science Foundation of China, grant number 42177119; and National Natural Science Foundation of China, grant number 51978440.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The data presented in this study are available on request from the corresponding author via e-mail: hzhang@dhu.edu.cn.

Acknowledgments: We thank the funders, thank the helpful comments from the editors and reviewers, and thank the contributions of all authors.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Duan, X.Y.; Yin, P.; Xie, Y.Q.; Cao, K.; Qiu, J.D.; Li, M.N.; Li, X. Advancements in the study of shallow gas in the coastal waters of China. *Mar. Geol. Quat. Geol.* **2024**, *44*, 183–196. [CrossRef]
2. Zhang, F.; Jia, P.F.; Wang, Y.; Chen, K.W. Geological occurrence characteristics and engineering hazards of ultra shallow-buried gases in Ningming Basin. *J. Eng. Geol.* **2023**, *31*, 1105–1115. [CrossRef]
3. Zhou, Q.K.; Sun, Y.F.; Song, Y.P.; Li, X.S.; Dong, L.F.; Song, B.H.; Du, X.; Wang, C.; Xiu, Z.X. Distribution of shallow gas at an offshore platform site in Bohai Bay and its genetic mechanism. *Geol. Bull. China* **2021**, *40*, 298–304.
4. Song, J.; Liu, L.L.; Liu, T.; Zhang, Y.C.; Yang, L.; Wan, Y.Z. Deformation and seepage characteristics of marine gassy soil during high-stress loading and unloading. *Chin. J. Theor. Appl. Mech.* **2024**; *in press*. [CrossRef]
5. Ye, Z.B.; Li, J.R.; Ye, Q.Y.; Chen, X.X.; Zhou, W.T. Experimental study on consolidation and settlement characteristics of fine-grained gassy soil. *J. Ground Improv.* **2022**, *4*, 472–478. [CrossRef]
6. Tong, S.J.; Huang, M.S.; Shi, Z.H.; Wang, B. Experimental study of mechanical properties of gassy sand and state-dependent constitutive modeling. *Chin. J. Geotech. Eng.* **2024**; *in press*. [CrossRef]
7. Kong, L.W.; Zhong, F.J.; Guo, A.G.; Wang, Y. Experimental study of stress path of shallow gassy sand of Hangzhou Bay. *Rock Soil Mech.* **2009**, *30*, 2209–2214. [CrossRef]
8. Guo, A.G.; Kong, L.W.; Chen, J.B.; Zhong, F.J. On engineering characteristics of shallow biogenetic gassy sand by piezocone test. *Rock Soil Mech.* **2007**, *28*, 1539–1543. [CrossRef]
9. Wang, Y.; Kong, L.W.; Guo, A.G. Analysis of one-dimensional deformation of shallow gassy stratum caused by controlling gas release. *Rock Soil Mech.* **2011**, *32*, 241–246. [CrossRef]
10. Xu, M.B.; Chen, C.; Wang, Y.; Yan, J.B.; Yu, S.; Zhang, J.J. Undrained shear mechanical behaviors of dense gassy sand. *J. Rock Mech. Geotech. Eng.* **2024**; *in press*. [CrossRef]
11. Wang, H.; Zhang, Y.; Cai, G.J.; Liang, Y.L.; Liu, T.; Chen, J. Study on the CPTu inversion methods for strength and consolidation parameters of gassy soil with different gas contents. *Appl. Ocean. Res.* **2024**, *146*, 103960. [CrossRef]
12. Vega-Posada, C.A.; Finno, R.J.; Zapata-Medina, D.G. Effect of gas on the mechanical behavior of medium-dense sands. *J. Geotech. Geoenviron. Eng.* **2014**, *140*, 14014063. [CrossRef]
13. Hong, Y.; Chen, X.Y.; Wang, L.Z.; Yang, B.; Sanchez, M. Distinct undrained cyclic behavior of fine-grained gassy soil at various initial pore water pressures. *J. Geotech. Geoenviron. Eng.* **2020**, *147*, 06020027. [CrossRef]

14. Wang, Y.; Kong, L.W.; Wang, Y.L.; Wang, M.; Cai, K.J. Deformation analysis of shallow gas-bearing ground from controlled gas release in Hangzhou Bay of China. *Int. J. Geomech.* **2018**, *18*, 04017122. [CrossRef]
15. Yang, G.C.; Liu, Y.; Chen, P.P. Simulation analysis of temperature effects on the shear behavior of gassy sand. *Geofluids* **2022**, *2022*, 5602833. [CrossRef]
16. Liu, L.L.; Liu, T.; Wu, C.; Bu, Q.T.; Li, C.F.; Zhang, Y.C.; Wu, B.S. A multi-orientation system for characterizing microstructure changes and mechanical responses of fine-grained gassy sediments associated with gas hydrates. *Rev. Sci. Instrum.* **2024**, *95*, 073704. [CrossRef] [PubMed]
17. Sultan, N.; De Gennaro, V.; Puech, A. Mechanical behaviour of gas-charged marine plastic sediments. *Géotechnique* **2012**, *62*, 751–766. [CrossRef]
18. Wang, Y.; Mu, K.; Wang, Y.L. Experimental study on strength and deformation characteristics of the gassy muddy clay with thin silt interlayers. In *IACGE 2013: Challenges and Recent Advances in Geotechnical and Seismic Research and Practices*; Hu, J., Ma, J., Meneses, J., Qui, T., Yu, X., Zeng, X., Eds.; American Society of Civil Engineers: Reston, VA, USA, 2013; pp. 523–530. [CrossRef]
19. Hyodo, M.; Li, Y.; Yoneda, J.; Nakata, Y.; Yoshimoto, N.; Nishimura, A.; Song, Y. Mechanical behavior of gas-saturated methane hydrate-bearing sediments. *J. Geophys. Res. Solid Earth* **2013**, *118*, 5185–5194. [CrossRef]
20. Yoneda, J.; Kida, M.; Konno, Y.; Jin, Y.; Morita, S.; Tenma, N. In situ mechanical properties of shallow gas hydrate deposits in the deep seabed. *Geophys. Res. Lett.* **2019**, *46*, 14459–14468. [CrossRef]
21. Mitchell, J.K.; Soga, K. *Fundamentals of Soil Behavior*, 3rd ed.; John Wiley & Sons: New York, NY, USA, 2005; p. 409.
22. Wheeler, S. The Stress-Strain Behaviour of Soils Containing Gas Bubbles. Ph.D. Thesis, University of Oxford, Oxford, UK, 1986.

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Characterizing the Interaction Between Asphalt and Mineral Fillers in Hot Mix Asphalt Mixtures: A Micromechanical Approach

Shuang Wang ^{1,2,3}, Zhichen Wang ^{1,3,4,*}, Ankang Yu ⁵, Huanan Yu ⁴, Zhongming He ⁴, Xiangzhu Meng ⁶, Zhi Gong ¹, Deqing Guan ¹ and Fuli Zhang ¹

¹ School of Computer Science and Engineering, Hunan University of Information Technology, Changsha 410151, China; wangshuang85421@163.com (S.W.); gongzhi1@hnuit.edu.cn (Z.G.); guan491596452@163.com (D.G.); zhangfuli@hnuit.edu.cn (F.Z.)

² Faculty of Education, Universiti Kebangsaan Malaysia, Bangi 43600, Malaysia

³ Intelligent Engineering Technology Research Center, Harbin Cambridge University, Harbin 150069, China

⁴ College of Transportation Engineering, Changsha University of Science and Technology, Changsha 410114, China; huanan.yu@csust.edu.cn (H.Y.); hezhongming@csust.edu.cn (Z.H.)

⁵ Jimo Highway Development Center of Qingdao City, Qingdao 266000, China; yuankang18153@163.com

⁶ Department of Road and Bridge Engineering, Liaoning Provincial College of Communications, Shenyang 110122, China; mxz42423699@163.com

* Correspondence: zhichenwang@163.com

Abstract: Asphalt mastic serves as a critical binding material in hot mix asphalt mixtures, significantly influencing the performance and durability of asphalt pavements. The interaction between asphalt and mineral fillers directly affects the binding properties of the mastic. In this study, the adsorbed asphalt film thickness was used as an indicator to evaluate the interaction between asphalt and mineral fillers. A micromechanical approach was proposed to calculate this thickness, and the results were compared using the Hashin model, the Mori–Tanaka model, and the generalized self-consistent model. The results demonstrate that the adsorbed asphalt film thickness, as determined using the micromechanical approach, ranged from 0.01 to 0.37 μm . The Hashin model was found to provide the most accurate characterization of the interaction between the asphalt and the mineral fillers. The order of adsorbed asphalt film thickness was as follows: coal gangue asphalt mastic > limestone asphalt mastic > fly ash asphalt mastic. Higher concentrations of acidic SiO_2 in the mineral fillers resulted in a weaker interaction between the asphalt and the fillers. When the temperature was below the softening point of the asphalt, the interaction strength decreased as frequency increased. Conversely, when the temperature exceeded the softening point, the interaction strength increased with frequency. The effect of temperature on the interaction capability was further influenced by the characteristics of the mineral fillers. The micromechanical-based method proposed in this study eliminates the dependency of the evaluation indicator on the volume fraction of mineral fillers, thereby providing a more accurate characterization of the interaction between asphalt and fillers. This approach provides a theoretical foundation to guide the design of asphalt mixtures.

Keywords: asphalt mastic; interaction; micromechanical model; adsorbed asphalt film thickness; mineral filler

1. Introduction

Asphalt pavements offer several advantages, such as enhanced driving comfort, reduced noise levels, minimal dust, and ease of maintenance. Consequently, asphalt mixtures

are widely used as the primary material for highway pavement construction [1,2]. However, asphalt pavements are susceptible to various forms of distress, including cracking, loosening, and particle loss. These issues not only shorten the service life, but also pose significant challenges in achieving the designed lifespan of pavements [3,4]. Therefore, improving the performance and extending the service life of asphalt pavements has become a central focus of research.

The service life of asphalt pavements is closely related to the performance of asphalt mixtures. Asphalt mixtures are multiphase composites composed of asphalt, aggregates, and mineral fillers blended in specific proportions. The asphalt mastic, consisting of asphalt and mineral fillers, serves as a binder in asphalt mixtures. Previous studies have identified asphalt mastic as the most vulnerable component in asphalt mixtures [5]. The properties of asphalt mastic significantly influence the high-temperature stability, low-temperature crack resistance, and water resistance of asphalt mixtures [6,7]. The performance of asphalt mastic is primarily determined by the interaction between asphalt and mineral filler. Therefore, the characterization of the interactions between asphalt and mineral fillers, along with the exploration of their influencing factors, is critical for enhancing the performance of asphalt mastic.

The rheological properties of asphalt mastic are significantly influenced by the addition of mineral fillers. Early studies assessed the effects of mineral fillers on asphalt properties using indicators such as softening point, penetration, ductility, and viscosity. While these methods provide a clear understanding of how mineral fillers affect asphalt properties, they do not adequately characterize or evaluate the interaction between asphalt and mineral filler [8,9]. In recent years, the assessment of this interaction has increasingly relied on a series of interaction indices based on rheological properties, which are primarily categorized into three types.

Phase-Angle-Based Indices: The Luis Ibrarra-*A* index evaluates the interaction between asphalt and mineral fillers by measuring energy dissipation at the interface through the loss factor ($\tan\delta$) [10]. Ziegel et al. proposed that mineral filler adsorbs free asphalt, forming an interfacial layer that enhances its effective volume fraction. The Ziegel-*B* index quantifies this interaction by adjusting the filler volume fraction based on the phase angle [11].

Viscosity-Based Indices: Einstein's model describes a linear relationship between spherical particle volume fraction and solution viscosity, applicable only for fractions below 0.1 and neglecting particle-solution interactions [12]. Chong et al. developed a semi-empirical viscosity equation that incorporates interaction effects [13]. Tan et al. introduced the composite viscosity coefficient $\Delta\eta^*$, which reflects asphalt-filler interactions but is also influenced by filler particle size and shape [14].

Complex Shear-Modulus-Based Indices: Tan et al. proposed the complex shear modulus coefficient ΔG^* to characterize asphalt-filler interactions [15]. Ziegel and Romanov further developed a coefficient, denoted as *C*, to assess the interaction between asphalt and mineral fillers. This coefficient is referred to as the Palierne-*C* index [11].

Tan et al. concluded that the Ziegel-*B* index is more sensitive and physically significant than the Luis Ibrarra-*A*, ΔG^* , and $\Delta\eta^*$ indices [14]. Liu et al. found that ΔG^* and the Palierne-*C* index better reflect interaction capability than the Einstein coefficient but show lower sensitivity to changes in asphalt properties [16]. Guo's analysis indicated that all three indices are influenced by filler volume fraction, with the Palierne-*C* index exhibiting the least dependence [17].

From the above analysis, it is clear that the applicability of interaction evaluation methods based on rheological properties varies under different conditions. Moreover, these methods also exhibit dependence on the volume fraction of the mineral filler. Guo analyzed

the variations in three indicators—Luis Ibrarra-*A*, Ziegel-*B*, and Palierne-*C*—relative to the volume fraction of mineral filler. The results indicated that all three indicators are influenced by the volume fraction of mineral filler, with the Palierne-*C* index showing the least dependence [17]. Since these interaction indicators are calculated at a single volume fraction of mineral filler, this dependency affects the accuracy of the interaction evaluation.

In addition to rheology-based evaluation indices, relevant standards have also established parameters for assessing the interaction between bitumen and mineral fillers, such as the Bitumen Capacity Index (BCI). The BCI is a key parameter for quantifying the interaction between bitumen and mineral fillers, as defined in the European Standard EN 13179-2 [18]. The calculation of BCI typically involves determining the bitumen adsorption, viscosity, and specific surface area of the filler, as well as its chemical reactivity. However, these standard methods have certain limitations. They require multiple steps and sophisticated equipment, which can be time-consuming and resource-intensive [19]. Moreover, these methods primarily focus on adsorption and chemical reactivity but may not completely capture the full range of physical and mechanical interactions between bitumen and fillers under operational conditions.

According to the research of L.A. Repin on the interaction between asphalt and mineral filler, physicochemical interactions occur between asphalt and mineral filler during high-temperature mixing. Asphalt undergoes a rearrangement of its chemical components on the surface of the mineral filler, forming a structural asphalt layer [20]. The thickness of this layer is referred to as the adsorbed asphalt film thickness. The thickness of the adsorbed asphalt film is directly proportional to the intensity of the physicochemical interaction between the asphalt and the mineral filler. Therefore, compared to rheological property-based evaluation indicators, the adsorbed asphalt film thickness provides a more accurate representation of the interaction capability between asphalt and mineral fillers.

To measure the thickness of the adsorbed asphalt film on mineral filler, Tan et al. employed Differential Scanning Calorimetry (DSC) to determine the glass transition temperature of asphalt mastic. The thickness of the adsorbed asphalt film was calculated based on changes in specific heat capacity during the glass transition process. Using this method, the calculated thickness ranged from 0.02 to 5.28 nm [20]. However, DSC is typically applicable to homogeneous materials, where thermal properties such as specific heat capacity and thermal conductivity are uniform throughout the sample. For asphalt mastic, which is a heterogeneous material, the results obtained from DSC may not accurately reflect the true thermal behavior of the material. Wide-Angle X-ray Diffraction (WAXD) offers several advantages over DSC for analyzing the crystalline structure of asphalt mastic. WAXD can directly observe the presence and arrangement of crystalline domains, providing detailed information on the crystalline phases and molecular orientation within the material. This technique is particularly suitable for heterogeneous materials like asphalt mastic, as it can provide spatially resolved information on the crystalline structure. Recent studies, such as the work by Pipintakos et al. [21], have demonstrated the feasibility of using WAXD to characterize the crystalline structure and its changes under different conditions, suggesting that WAXD could be a valuable tool for measuring the adsorbed asphalt film thickness.

Guo et al. utilized toluene to sequentially elute and separate free asphalt from structural asphalt, subsequently calculating the adsorbed asphalt film thickness, which was approximately 1 μm . However, the accuracy of these results may be influenced by the complexity of the elution experiments and measurement errors [22,23]. Therefore, precise determination of the adsorbed asphalt film thickness necessitates further in-depth research.

The road performance of asphalt mixtures is primarily influenced by the properties of the asphalt mastic, which result from the interactions between the asphalt and the mineral filler. However, the interactions between asphalt and mineral filler within the mastic, as

well as the reinforcement mechanisms of the mineral filler, remain subjects that require further investigation. The micromechanics of composite materials provides an effective approach for studying the interaction mechanisms among the various components of asphalt mastic [24,25].

Numerous scholars have extensively studied micromechanics models, including the Hashin model [26], the Mori–Tanaka (MT) model [27], the Generalized Self-Consistent (GSC) model [28], the Differential Scheme (DS) model [29], and the Periodic Structure (PS) model [30]. Among these, the Hashin model, the Mori–Tanaka (MT) model, and the generalized self-consistent (GSC) model are the most widely applied and foundational. The Hashin model is primarily designed for two-phase and multiphase heterogeneous materials [31], assuming that spherical inclusions are uniformly distributed in an isotropic homogeneous matrix, with no interaction between inclusion particles. It uses linear elastic variational principles to derive expressions for the elastic modulus, offering a concise formula with clear physical significance. The Mori–Tanaka model assumes that ellipsoidal inclusions of varying sizes are randomly distributed in the matrix and calculates the average stress within the matrix. This model effectively addresses the key theoretical challenges associated with applying Eshelby's equivalent inclusion principle under finite volume fraction conditions [32]. The generalized self-consistent model assumes spherical inclusions surrounded by a matrix shell embedded in an effectively infinite medium. In this model, the volume fraction of inclusions in the composite material is defined as the ratio of the inclusion to the matrix shell. Christensen and Lo proposed a three-phase generalized self-consistent inclusion model [28], and Shashidhar and Shenoy simplified the complex equations of the generalized self-consistent model through data analysis, providing a theoretical foundation for the use of micromechanics models in predicting the properties of asphalt mastic [31].

The reinforcement mechanisms of mineral filler in asphalt mastic mainly include volumetric filling reinforcement, physicochemical interaction reinforcement, and interparticle interaction reinforcement. Buttlar et al. proposed that micromechanics models can be employed to distinguish these three reinforcement mechanisms in asphalt mastic [33]. Shashidhar and Shenoy investigated the capability of the generalized self-consistent (GSC) model in predicting the dynamic mechanical behavior of asphalt mastic with mineral filler volume fractions of up to 31% [31]. The study found that the GSC model underestimates the effective shear modulus, primarily due to its incomplete representation of the volumetric reinforcement effect of mineral filler. Aigner et al. used BBR (Bending Beam Rheometer) and DSR (Dynamic Shear Rheometer) tests to determine the viscoelastic properties of asphalt. They proposed a multiscale model based on the Mori–Tanaka model to predict the viscoelastic behavior of asphalt mastic and examined the effects of voids and aggregates [33]. Kim conducted experiments on four different materials at mineral filler volume fractions of 5%, 10%, and 25%. The experimental results were compared with predictions from the Hashin model and the generalized self-consistent model. The study found that the predictions aligned more closely with the experimental results at lower mineral filler volume fractions, suggesting that interparticle and physicochemical interactions enhance the performance of asphalt–aggregate mixtures [34]. Underwood evaluated and compared 12 existing micromechanics models. Based on the limitations of these models, a new micromechanics model was proposed, incorporating physicochemical interactions. The results showed that this new model provides accurate predictions for asphalt mastic with mineral filler volume fractions of up to 40% [35].

In recent years, micromechanics models have been increasingly applied to calculate the thickness of adsorbed asphalt films. Buttlar et al. proposed a micromechanics-based method for determining the thickness of adsorbed asphalt films while investigating the

generalized self-consistent (GSC) model's ability to predict the mechanical properties of asphalt mastic. Using this method, the calculated thickness ranged from 0.02 to 0.1 μm [33]. Li et al. employed the Hashin model to calculate the thickness of the adsorbed asphalt films and evaluated the physicochemical interactions between asphalt and mineral filler based on the derived film thickness [36,37]. Utilizing micromechanics methods to calculate the thickness of adsorbed asphalt films effectively mitigates the influence of experimental and measurement errors, thereby improving the accuracy of the results. However, existing studies have predominantly relied on a single micromechanics model for these calculations. Further research is needed to assess the applicability of various micromechanics models for calculating the thickness of adsorbed asphalt films and to analyze the factors that influence this thickness in greater detail.

The research objectives of this paper are as follows:

- (1) To calculate the thickness of the adsorbed asphalt film using the Hashin, Mori–Tanaka (MT), and generalized self-consistent (GSC) models, and to perform a comparative analysis of the values obtained from these models to identify the most appropriate one.
- (2) To determine the critical volume fraction of mineral fillers using nonlinear fitting, and to refine the interaction evaluation indices based on micromechanical models, removing their dependence on the volume fraction of mineral fillers.
- (3) To analyze the influence of the chemical composition, testing frequency, and temperature of mineral fillers on the interaction between asphalt and mineral fillers through a micromechanical approach.

2. Calculation of Adsorbed Asphalt Film Thickness Based on Micromechanical Model

2.1. Hashin Model

The Hashin model, based on the variational principle of linear elasticity, utilizes an approximate approach to derive the elastic modulus expression for two-phase or multiphase heterogeneous materials. This model assumes that inclusions are spherical and uniformly distributed within an isotropic, homogeneous matrix, with no interaction between the inclusion particles. This configuration is illustrated schematically in Figure 1.

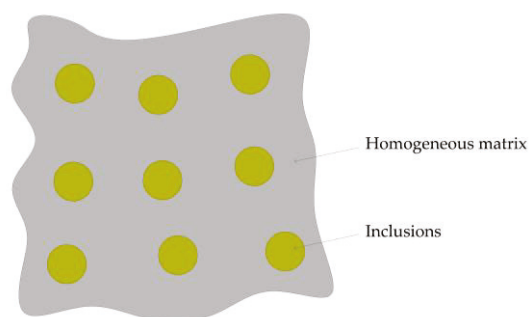


Figure 1. Schematic diagram of the Hashin model.

The effective shear modulus of the model is calculated using Equation (1):

$$\frac{G_c}{G_m} = 1 + \frac{15(1 - \nu_m)(G_f/G_m - 1)\varphi_f}{7 - 5\nu_m + 2(4 - 5\nu_m)[G_f/G_m - (G_f/G_m - 1)\varphi_f]} \quad (1)$$

where G_c represents the complex shear modulus of the asphalt mastic, G_m is the complex shear modulus of the asphalt, G_f is the complex shear modulus of the mineral filler, and ν_m denotes the Poisson's ratio of the asphalt. When rigid mineral filler particles are

dispersed in a viscoelastic medium, it is reasonable to assume that $G_f \gg G_m$, implying that $G_m/G_f \gg 1$ [31]. Under this assumption, Equation (1) can be simplified to Equation (2):

$$\frac{G_c}{G_m} = 1 + \frac{15(1 - v_m)\phi_f}{2(4 - 5v_m)(1 - \phi_f)} \quad (2)$$

By substituting the experimental value of G_c/G_m into Equation (2), the effective volume fraction of the mineral filler (ϕ_{eff}) can be determined, as expressed in Equation (3):

$$\phi_{\text{eff}} = \frac{2(G_c/G_m - 1)(4 - 5v_m)}{15(1 - v_m) + 2(G_c/G_m - 1)(4 - 5v_m)} \quad (3)$$

Buttler et al. [33] proposed a micromechanical approach to calculate the thickness of the adsorbed asphalt film, as shown in Equation (4):

$$d = \frac{\phi_{\text{eff}} - \phi_f}{\phi_f \times G_s \times S} \quad (4)$$

where d represents the adsorbed asphalt film thickness, ϕ_{eff} is the effective volume fraction of the mineral filler, ϕ_f is the volume fraction of the mineral filler, G_s is the apparent density of the mineral filler, and S is the specific surface area of the mineral filler. By incorporating ϕ_{eff} into Equation (4), the thickness of the adsorbed asphalt film based on the Hashin model can be accurately determined.

2.2. Mori–Tanaka Model

In 1973, Mori and Tanaka resolved the fundamental theoretical issues associated with the application of Eshelby's equivalent inclusion principle in finite volume fractions by calculating the average stress within the matrix. Since then, the Mori–Tanaka model has been extensively employed to predict the effective properties of various composite materials [32]. This model assumes that the inclusions are ellipsoidal particles with random orientations and varying sizes, as illustrated in Figure 2.

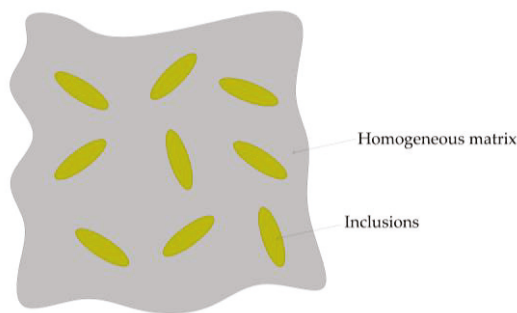


Figure 2. Schematic diagram of the Mori–Tanaka model.

When applying the Mori–Tanaka model to predict the effective shear modulus of asphalt mastic, Underwood et al. derived the following formula [35]:

$$\frac{G_c}{G_m} = 1 + \frac{5\phi_f(G_f/G_m - 1)}{5 + 2(1 - \phi_f)(G_f/G_m - 1)} \quad (5)$$

Considering the mineral filler particles as rigid inclusions, i.e., $G_f/G_m \gg 1$ [33], Equation (5) can be simplified to

$$\frac{G_c}{G_m} = 1 + \frac{5\phi_f}{2(1 - \phi_f)} \quad (6)$$

By substituting the experimental value of G_c/G_m into Equation (6), the effective volume fraction of the mineral filler (ϕ_{eff}) can be determined as follows:

$$\phi_{\text{eff}} = \frac{2(G_c/G_m - 1)}{3 + 2(G_c/G_m)} \quad (7)$$

The thickness of the adsorbed asphalt film based on the Mori–Tanaka model can be calculated by incorporating ϕ_{eff} into Equation (4).

2.3. The Generalized Self-Consistent Model

Christensen et al. proposed the generalized self-consistent (GSC) model, which is a three-phase model comprising inclusions, a matrix shell, and an effective medium. In this model, the proportion of inclusions and the matrix shell correspond precisely to the volume fraction of inclusions in the composite material [28], as illustrated in Figure 3.

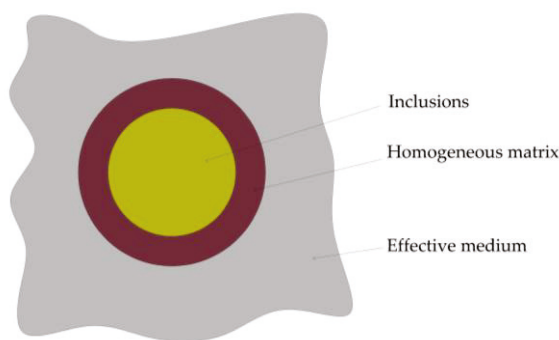


Figure 3. Schematic diagram of the generalized self-consistent model.

The effective shear modulus of the composite material is given by

$$A(G_c/G_m)^2 + B(G_c/G_m) + C = 0 \quad (8)$$

where A , B , and C are complex functions related to the ratio of the shear moduli of the mineral filler and asphalt (G_m/G_f).

Shashidhar et al. simplified the generalized self-consistent model for application in asphalt mastic by considering the condition $G_f \gg G_m$ [33]. This simplification resulted in a micromechanical prediction model for the shear modulus of asphalt mastic, expressed as

$$A_1(G_c/G_m)^2 + B_1(G_c/G_m) + C_1 = 0 \quad (9)$$

with the following parameters:

$$A_1 = \left(\frac{8(4 - 5v_m)(7 - 10v_m)\phi_f^{10/3} - 50(7 - 12v_m + 8v_m^2)\phi_f^{7/3} + 252\phi_f^{5/3} - 50(7 - 12v_m + 8v_m^2)\phi_f + 8(4 - 5v_m)(7 - 10v_m)}{252\phi_f^{5/3} - 50(7 - 12v_m + 8v_m^2)\phi_f + 8(4 - 5v_m)(7 - 10v_m)} \right) \quad (10a)$$

$$B_1 = \left(\frac{-4(1 - 5v_m)(7 - 10v_m)\phi_f^{10/3} + 100(7 - 12v_m + 8v_m^2)\phi_f^{7/3} - 504\phi_f^{5/3} + 150(3 - v_m)v_m\phi_f + 6(4 - 5v_m)(15v_m - 7)}{504\phi_f^{5/3} + 150(3 - v_m)v_m\phi_f + 6(4 - 5v_m)(15v_m - 7)} \right) \quad (10b)$$

$$C_1 = \left(\frac{4(5v_m - 7)(7 - 10v_m)\phi_f^{10/3} - 50(7 - 12v_m + 8v_m^2)\phi_f^{7/3} + 252\phi_f^{5/3} + 25(v_m^2 - 7)\phi_f - (7 + 5v_m)(8 - 10v_m)}{252\phi_f^{5/3} + 25(v_m^2 - 7)\phi_f - (7 + 5v_m)(8 - 10v_m)} \right) \quad (10c)$$

where v_m denotes the Poisson's ratio of the asphalt.

The effective volume fraction of the mineral filler (ϕ_{eff}) can be determined by substituting the measured value of G_c/G_m into Equations (9) and (10). Using the obtained ϕ_{eff} ,

the thickness of the adsorbed asphalt film based on the generalized self-consistent model can then be accurately calculated through Equation (4).

3. Experimental Methods and Model Parameters

3.1. Experimental Methods

The asphalt used in this study is Liaohu 90# base asphalt, sourced from Panjin, Liaoning Province, China. The basic properties of the asphalt are summarized in Table 1.

Table 1. Basic properties of asphalt.

Index	Penetration at 25 °C (0.1 mm)	Softening Point (°C)	Ductility at 5 °C (cm)	Flash Point (°C)	Density at 15 °C (g/cm ³)
Test results	80	48	8.6	254	1.003
Specification requirements (JTG F40-2004) ¹	80–100	≥44	/	≥254	As-measured record

¹ JTG F40-2004 is a national highway engineering standard in China that provides technical requirements and guidelines for the construction of asphalt pavements.

Three types of mineral fillers were selected for this study: limestone filler, fly ash, and coal gangue filler. Natural limestone was sourced from Zhengzhou, Henan, China, while fly ash was obtained from the Zhengzhou Power Plant, and coal gangue filler was derived from crushed unburned coal gangue from the Zhengzhou Coal Mine. The appearance of these fillers is shown in Figure 4. The specific surface area of the three fillers was measured using the Brunauer–Emmett–Teller Surface Area Analyzer (Autosorb-iQ, Quantachrome Instruments, Boynton Beach, FL, USA). The basic properties of the fillers are presented in Table 2.

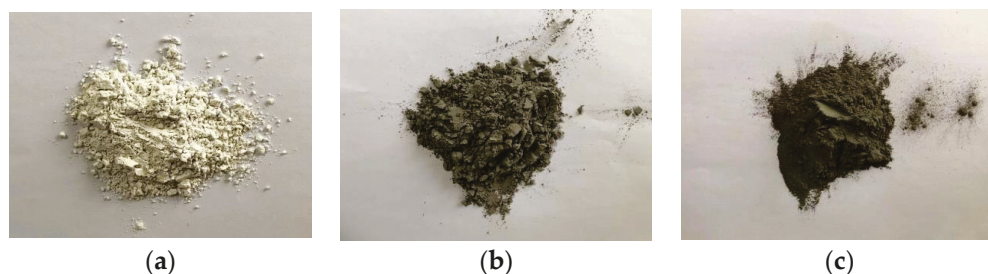


Figure 4. Pile appearance of different fillers: (a) limestone filler; (b) fly ash; (c) coal gangue filler.

Table 2. Basic properties of fillers.

Index	Limestone Filler	Fly Ash	Coal Gangue Filler
Apparent density (g/cm ³)	2.971	1.896	1.901
Specific surface area (m ² /g)	0.911	2.829	2.681

A laser particle size analyzer was employed to sequentially test the three types of mineral powders. The average of three measurements was taken to plot the particle size distribution curves, as shown in Figure 5. The results indicate that among the three types of mineral powders, the fly ash powder exhibits the finest particle size, with 90% of the particles having a diameter below 26.942 µm. The particle sizes of the three mineral powders, in ascending order, are as follows: fly ash < limestone filler < coal gangue filler.

Three types of fillers were selected and mixed with 90# base asphalt to prepare the asphalt mastics. The filler-to-binder (F/B) ratios were set at 0.4, 0.8, 1.2, 1.6, and 2.0. The resulting mastics were designated as limestone asphalt mastic (LAM), fly ash asphalt mastic

(FAM), and coal gangue asphalt mastic (CAM). The preparation process of the asphalt mastic was as follows: The fillers were first dried in an oven at 165 °C for 2 h to remove any residual moisture. Subsequently, the dried fillers were cooled to room temperature in a desiccator to ensure thermal stability prior to mixing. Meanwhile, the base asphalt was heated to 140 °C to achieve optimal fluidity for uniform mixing. After weighing the fillers and asphalt according to the predetermined filler-to-binder ratios, the components were mixed in a preheated iron pan to ensure thorough dispersion and adhesion between the fillers and asphalt binder. The mixture was then stirred with an electric stirrer at a speed of 300 RPM and a temperature of 150 °C for 15 min, until homogeneity was achieved and no air bubbles were present on the surface of the asphalt mastic.

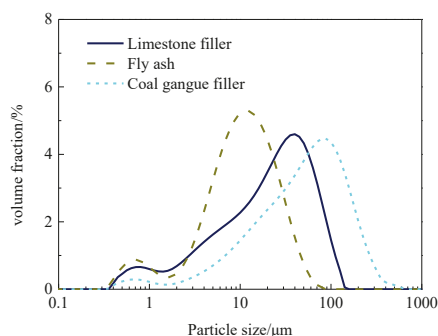


Figure 5. Particle size distribution of different types of mineral fillers.

A Bohlin Gemini rheometer was utilized to conduct temperature–frequency scanning tests on the asphalt and asphalt mastic samples to measure their complex shear modulus values. An 8 mm diameter parallel plate was used, with a sample thickness of 2 mm. The loading rate ranged from 0.628 to 125.7 rad/s, corresponding to frequencies from 0.1 to 20 Hz. The tests were performed at temperatures of 28 °C, 40 °C, and 52 °C, with a loading strain of 0.1%.

3.2. Model Parameters

In the micromechanical model, the complex shear modulus of both the asphalt and asphalt mastic was determined through Dynamic Shear Rheometer (DSR) tests. The volume fraction of filler in the asphalt mastic was calculated based on the filler-to-binder (F/B) ratio, with the results presented in Table 3.

Table 3. Volume fraction of different fillers in asphalt mastic.

Filler Type	Volume Fraction in Asphalt Mastic				
	F/B = 0.4	F/B = 0.8	F/B = 1.2	F/B = 1.6	F/B = 2.0
Limestone filler	11.89%	21.26%	28.82%	35.06%	40.30%
Fly ash	17.46%	29.73%	38.82%	45.83%	51.40%
Coal gangue filler	17.42%	29.67%	38.76%	45.77%	51.33%

Di Benedetto et al. observed that the Poisson’s ratio of asphalt varies slightly with frequency. The range is reported as 0.48–0.5 [38]. For the purpose of simplification, this study assumes a constant Poisson’s ratio of 0.49, disregarding minor frequency-dependent variations.

3.3. Critical Volume Fraction of Mineral Filler in Asphalt Mastic

To investigate the interaction mechanism between asphalt and mineral filler, Faheem and Bahia classified the variation in the complex shear modulus (G^*) of asphalt mastic with the volume fraction of mineral filler into two distinct regions: the “dilution zone” and the

“aggregation zone” [39]. In the dilution zone, where the volume fraction of mineral filler is relatively low, asphalt can completely encapsulate the mineral filler particles, preventing any contact between them. The reinforcement effect of the mineral filler in this region primarily results from the volumetric filling effect and the physicochemical interactions between the mineral filler and asphalt. As the volume fraction of mineral filler increases to a specific threshold, the particles, although still largely encapsulated by asphalt, begin to make contact with each other. At this point, a network structure starts to form among the particles, and the corresponding volume fraction is defined as the critical volume fraction. In the aggregation zone, when the mineral filler volume fraction exceeds the critical value, clustering of the mineral filler particles occurs. Asphalt can no longer fully encapsulate the particles, and significant interactions between the particles take place. Therefore, accurately determining the critical volume fraction of mineral filler is crucial. Additionally, when studying the interaction between asphalt and mineral filler, the mineral filler volume fraction should be maintained within the critical volume fraction range.

To accurately calculate the critical volume fraction of mineral filler and establish the research range for the interaction between asphalt and mineral filler, this study employed the asphalt mastic model proposed by Faheem and Bahia [39]. This model predicts the hardening effect of mineral filler on asphalt based on the intrinsic properties of asphalt and filler. The governing equations of the model are expressed as follows:

$$G_r = G_1 + a_1(\phi - \phi_c) + G_2(a_2 - a_1) \ln\{1 + \exp[(\phi - \phi_c)/G_2]\} \quad (11)$$

$$G_r = \frac{G_c}{G_m} \quad (12)$$

where G_r is the shear modulus ratio of asphalt mastic to asphalt (dimensionless); G_c is the complex shear modulus of asphalt mastic (kPa); G_m is the complex shear modulus of asphalt (kPa); ϕ is the volume fraction of mineral filler (%); ϕ_c is the critical volume fraction of mineral filler (%); and G_1 , G_2 , a_1 , and a_2 are fitting parameters (dimensionless).

Equations (11) and (12) served as the fitting functions for determining the critical volume fraction of mineral filler. Nonlinear fitting was performed on the experimental data of the shear modulus ratio of the asphalt mastic at various mineral filler volume fractions. The fitting results, along with the goodness of fit, are presented in Table 4.

Table 4. Fitting results and goodness of fit for critical volume fraction of filler.

Filler Type	Fitting Parameters					Correlation Coefficient R^2
	G_1	ϕ_c	G_2	a_1	a_2	
Limestone filler	2.36	30.65%	0.62	0.03	0.45	0.916
Fly ash	1.83	44.96%	5.28	2.33	2.83	0.981
Coal gangue filler	4.29	41.15%	3.06	0.05	4.55	0.998

As shown in Table 4, the critical volume fraction (ϕ_c) of mineral filler in different types of asphalt mastic ranges from 30% to 45%, aligning well with findings from previous studies [40]. This consistency validates the reliability of the results obtained through the fitting process using Equations (11) and (12).

Specifically, the critical volume fractions for limestone asphalt mastic, fly ash asphalt mastic, and coal gangue asphalt mastic are 30.65%, 44.96%, and 41.15%, respectively. To minimize the influence of interactions between mineral filler particles on the analysis, subsequent investigations into the interaction mechanisms between asphalt and mineral filler should be restricted to the critical volume fraction ranges identified above.

4. Results and Discussion

4.1. Dynamic Shear Rheometer (DSR) Test Results

The variation in the complex shear modulus ($|G^*|$) with frequency is illustrated in Figure 6. The incorporation of mineral fillers significantly enhances the complex shear modulus of the matrix asphalt, thereby improving the high-temperature performance of the asphalt mastic. Moreover, as the filler-to-binder (F/B) ratio increases, the complex shear modulus exhibits a corresponding rise, consistent with the fundamental principles governing the reinforcing effect of mineral fillers on asphalt.

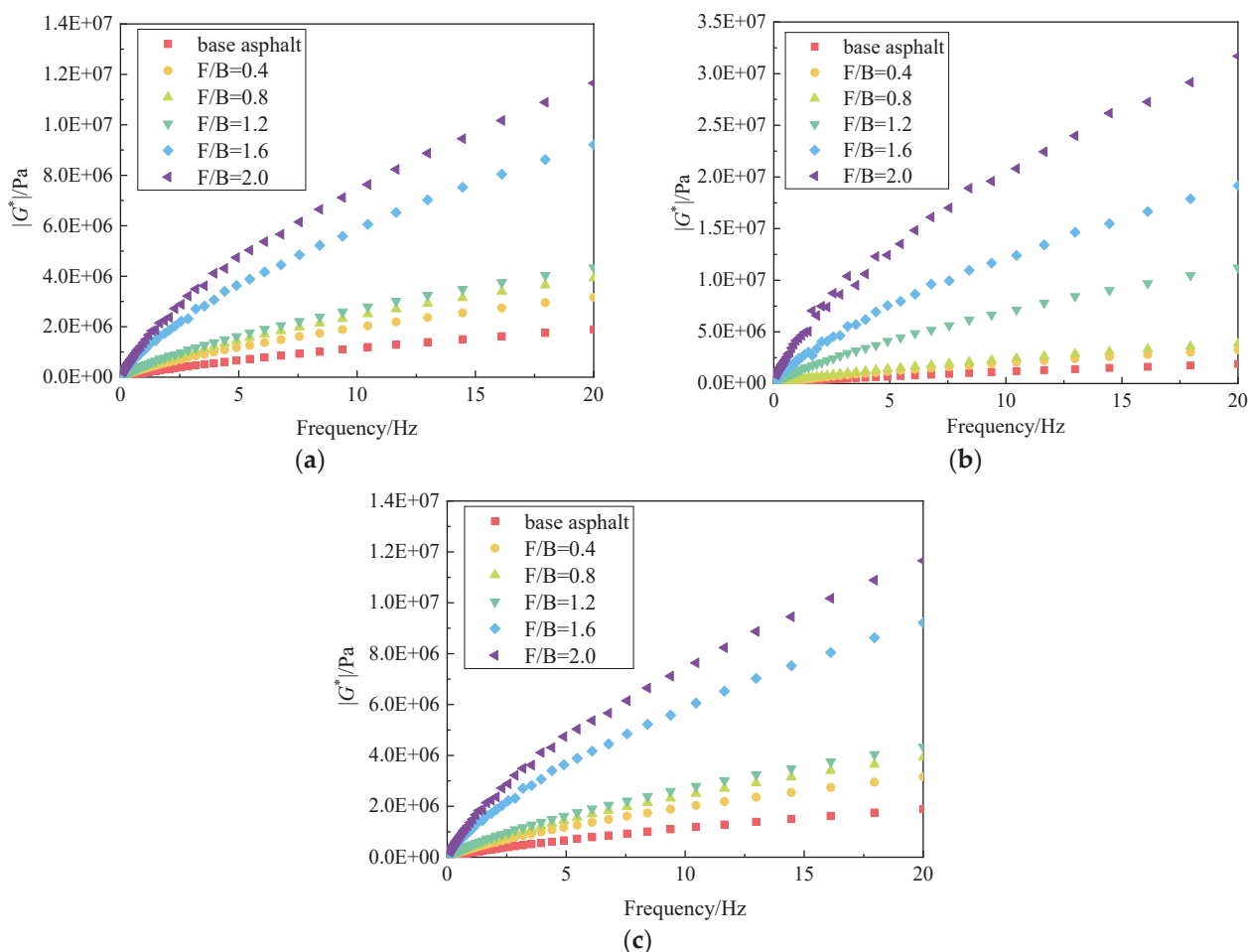


Figure 6. Complex shear modulus of asphalt mastic and base asphalt: (a) LAM; (b) FAM; (c) CAM.

4.2. Comparative Analysis of Adsorbed Asphalt Film Thickness Based on Different Micromechanical Models

The adsorbed asphalt film thickness was calculated using three micromechanical models to enable a comparative evaluation. The calculations were derived from Dynamic Shear Rheometer (DSR) test data. For consistency in the analysis, the film thickness was determined under controlled conditions at a temperature of 28 °C and a frequency of 5 Hz, while the effects of temperature and frequency variations were excluded. The results are presented in Figure 7.

As illustrated in Figure 7, the thickness of the adsorbed asphalt film between the asphalt and mineral filler ranges from 0.01 to 0.45 μm . The film thickness, calculated using the three micromechanical models, exhibits a similar trend with the filler-to-binder (F/B) ratio: initially decreasing and then increasing as the F/B ratio increases. This behavior is attributed to the reduction in asphalt content as the volume fraction of mineral filler

risers, resulting in a thinner adsorbed film. When the mineral filler volume fraction exceeds the critical value, particle interactions become significant, resulting in a rapid increase in the shear modulus of the asphalt mastic and, consequently, an increase in the calculated film thickness.

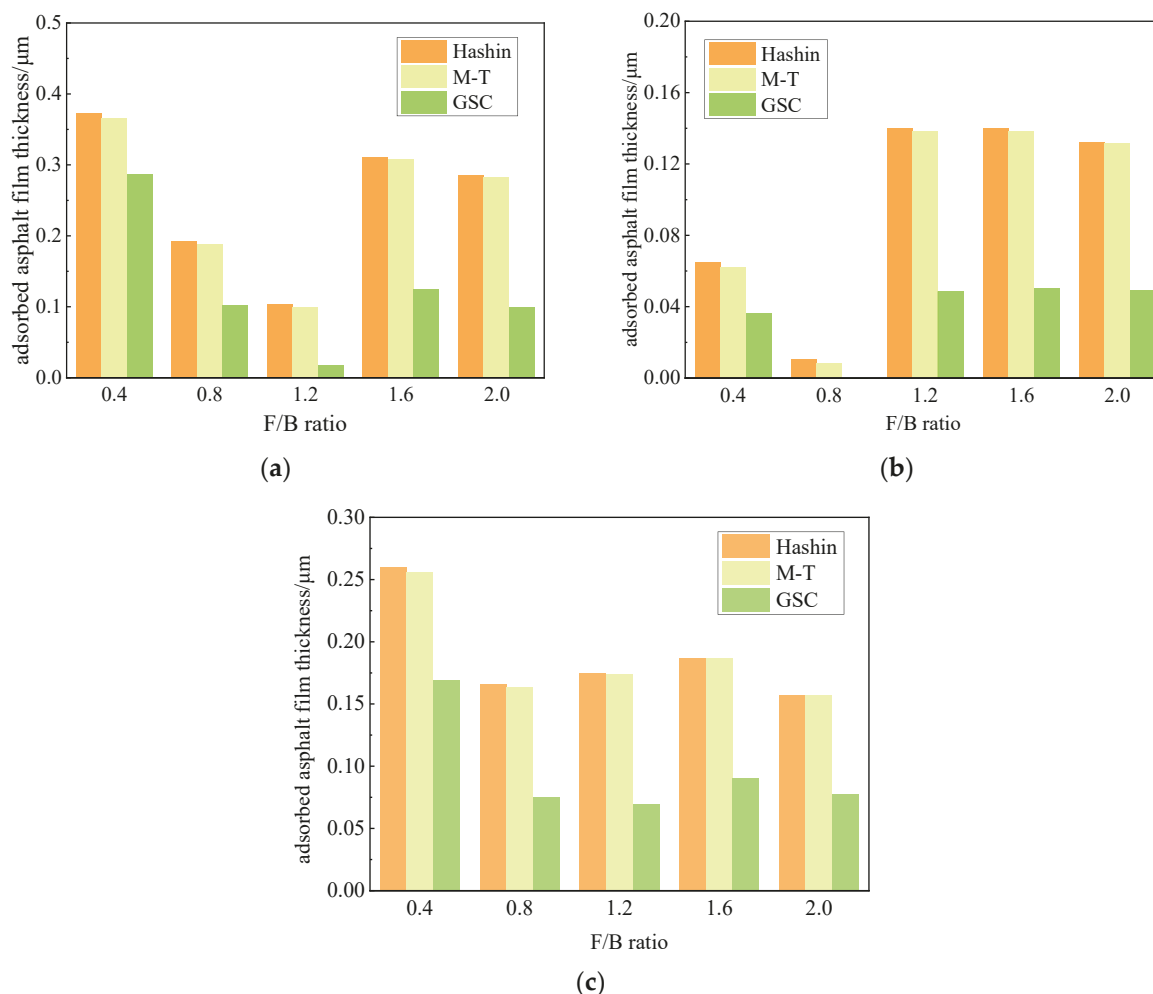


Figure 7. Adsorbed asphalt film thickness based on different micromechanical models: (a) LAM; (b) FAM; (c) CAM.

As shown in Figure 7b, the GSC model predicts a negative film thickness for FAM when the F/B ratio is 0.8. To investigate the cause of this discrepancy, the experimental G_c/G_m values for various asphalt mastics were compared with the predictions of micromechanical models, as illustrated in Figure 8.

Figure 8 indicates that for FAM with a mineral filler volume fraction of 29.73%, the experimental G_c/G_m value is lower than the GSC model's prediction. At this stage, the effective volume fraction (ϕ_{eff}) calculated using the GSC model is lower than the filler volume fraction (ϕ_f), resulting in the negative value observed in Figure 7b. Therefore, within a certain range of mineral filler content, the accuracy of the adsorbed asphalt film thickness predicted by the GSC model requires further validation.

The micromechanical models employed in this study—namely, the Hashin model, Mori–Tanaka model, and generalized self-consistent (GSC) model—provide valuable insights into the interaction between asphalt and mineral fillers. However, these models are based on certain assumptions that may not fully account for the complexities of the actual internal structure of asphalt mastic. For instance, asphalt mastic may contain internal voids, and the interface between the mineral filler and asphalt may not always be

perfectly bonded [24,25]. These factors are not explicitly considered in the models, leading to potential discrepancies between the model predictions and the actual internal structure of asphalt mastic. It should be noted that this limitation is not unique to these models; experimental methods for measuring the adsorbed asphalt film thickness also have inherent sources of error [15,17]. Achieving a completely accurate measurement of the adsorbed asphalt film thickness remains a challenge at the current stage of technology and requires further in-depth research. The trends in the adsorbed asphalt film thickness are analyzed through the application of these micromechanical models, rather than focusing on the absolute accuracy of the predictions. This approach enables meaningful conclusions to be drawn regarding relative interaction strengths, which are crucial for understanding the performance of asphalt mastics.

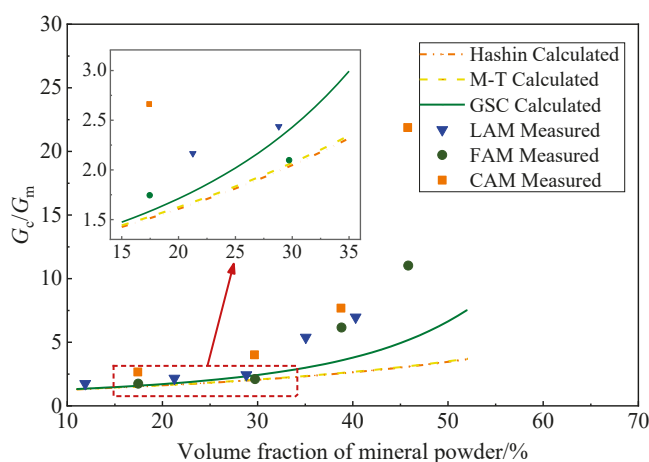


Figure 8. Comparison between measured value and calculated value of G_c/G_m for different mastics.

Figure 9 presents a comparison of film thickness predictions from three micromechanical models. It illustrates the relationship between the effective volume fraction of the mineral filler and the experimental G_c/G_m value. To eliminate the influence of factors such as mineral filler type, volume fraction, and temperature, the experimental G_c/G_m value for LAM with a mineral filler volume fraction of 11.89% at 28 °C was selected. In Figure 9, ϕ_1 , ϕ_2 , and ϕ_3 denote the differences between the effective volume fraction (ϕ_{eff}) and the filler volume fraction (ϕ_f) as predicted by the Hashin model, Mori–Tanaka model, and GSC model, respectively, for arbitrary experimental G_c/G_m values. According to Equation (4), for asphalt mastics with the same filler volume fraction, film thickness is determined by the difference between ϕ_{eff} and ϕ_f . Thus, ϕ_1 , ϕ_2 , and ϕ_3 serve as indicators of film thickness trends predicted by the three micromechanical models.

Within the critical volume fraction range of mineral filler defined in Section 3.3, before particles establish a contact network, their reinforcement effect on asphalt primarily results from volumetric filling and physicochemical interactions, without direct particle contact. The Hashin model captures the volumetric filling effect, whereas the Mori–Tanaka and GSC models incorporate inter-particle interactions to varying extents. The Mori–Tanaka model accounts for inter-particle interactions less comprehensively.

In Figure 9, shaded areas of varying intensity represent the magnitudes of volumetric filling enhancement and inter-particle interaction enhancement. The physicochemical interaction capability between mineral filler and asphalt is reflected in the thickness of the adsorbed asphalt film, which depends on the difference between ϕ_{eff} and ϕ_f . The lower shaded area (marked in the figure) represents the enhancement due to physicochemical interactions. The enlarged view of the Hashin and Mori–Tanaka model curves in Figure 9b shows that, compared to the Hashin model, the Mori–Tanaka model’s consideration of inter-

particle interactions results in a slight underestimation of physicochemical interactions between the asphalt and mineral filler. Consequently, the Mori–Tanaka model predicts a slightly smaller film thickness than the Hashin model.

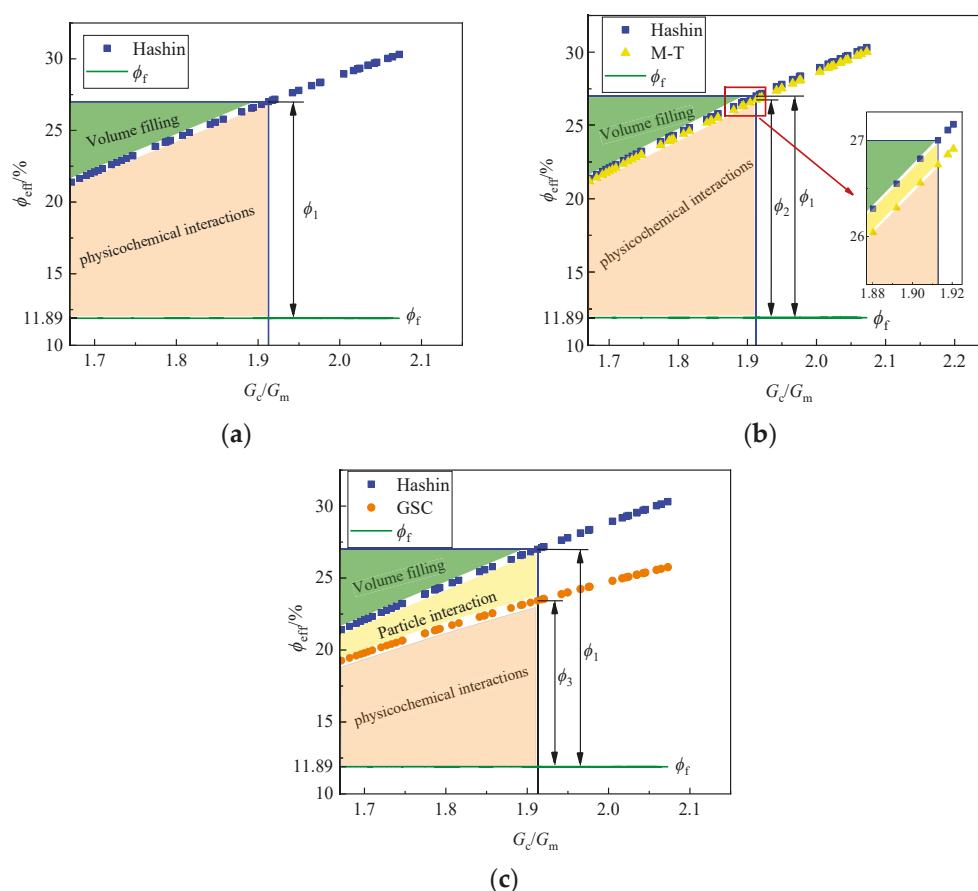


Figure 9. Relationship between the effective volume fraction of mineral filler and the experimental G_c/G_m value for three micromechanical models: (a) Hashin model only; (b) Hashin and Mori–Tanaka (M-T) models; (c) Hashin and GSC models.

A comparison of Figure 9a,c reveals that the GSC model significantly underestimates the physicochemical interactions between the asphalt and mineral filler due to its consideration of inter-particle interactions. As a result, the GSC model predicts a significantly smaller film thickness than the other two methods and may even yield negative values. Based on this analysis, the Hashin model is chosen to determine the adsorbed asphalt film thickness for evaluating asphalt–mineral filler interaction capability.

4.3. Improvement in Evaluation Indicators for the Interaction Between Asphalt and Mineral Filler Based on Hashin Model

As discussed in Section 4.2, while the Hashin model provides a more accurate representation of the physicochemical interactions between asphalt and mineral filler, the adsorbed asphalt film thickness remains influenced by the mineral filler volume fraction. Eliminating this dependency is essential to ensure a more reliable evaluation. To achieve this, further refinement of the calculation method for adsorbed asphalt film thickness is required to establish a more accurate and representative interaction evaluation indicator. This section introduces a nonlinear fitting method for calculating adsorbed asphalt film thickness, transforming it into an independent interaction evaluation indicator while removing its dependency on the mineral filler volume fraction. This refinement enables a

clearer distinction between the influence of material properties and external environmental factors on the interaction capability between asphalt and mineral fillers.

Based on the derivation results of the adsorbed asphalt film thickness presented in Section 2.1, the fitting formula for the interaction evaluation indicator based on the micromechanical model can be derived from Equations (3) and (4) as follows:

$$\frac{G_c}{G_m} = 1 + \frac{15\phi_f(1 - \nu_m)(d \times G_s \times S + 1)}{2(4 - 5\nu_m)[1 - \phi_f(d \times G_s \times S + 1)]} \quad (13)$$

where G_c/G_m is the experimental value of the shear modulus ratio of asphalt mastic. ϕ_f is taken as the independent variable, the measured value of G_c/G_m as the dependent variable, and d as the fitting parameter for nonlinear fitting. To ensure that the physicochemical interactions between the asphalt and mineral filler dominate without being affected by inter-particle interactions, the volume fraction of mineral filler should be restricted within the critical volume fraction range during fitting. The fitting process is shown in Figure 10.

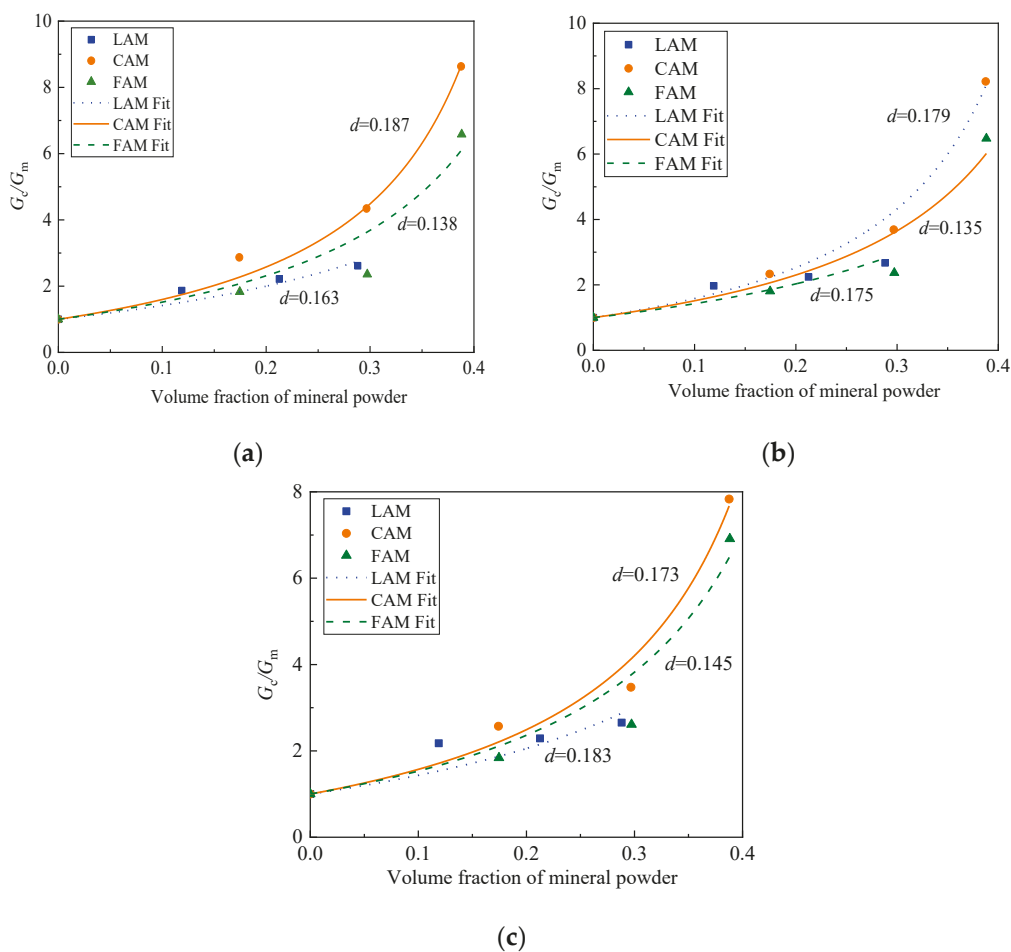


Figure 10. Fitting of adsorbed bitumen film thickness at different temperatures: (a) 28 °C; (b) 40 °C; (c) 52 °C.

The fitting results of the adsorbed asphalt film thickness for different asphalt mastics and their corresponding goodness of fit are presented in Table 5.

As presented in Table 5, the fitted values of film thickness d remain consistent across all temperatures and demonstrate a high goodness of fit. The adsorbed asphalt film thickness follows a clear trend: CAM exhibits the thickest film, followed by LAM, while FAM has the thinnest. This consistency indicates that the nonlinear fitting method effectively eliminates the influence of the original filler volume fraction. Consequently, this study employs the

fitted values of the adsorbed asphalt film thickness to assess the interaction capability between asphalt and mineral filler.

Table 5. Fitting results and goodness of fit for adsorbed film thickness for different types of asphalt mastic.

Temperature (°C)	LAM		FAM		CAM	
	Fitting Result	Correlation Coefficient R ²	Fitting Result	Correlation Coefficient R ²	Fitting Result	Correlation Coefficient R ²
28	0.163	0.888	0.138	0.897	0.187	0.989
40	0.175	0.844	0.135	0.899	0.179	0.989
52	0.183	0.826	0.145	0.924	0.173	0.977

4.4. Analysis of Factors Affecting the Interaction Capability Between Asphalt and Mineral Filler

4.4.1. Influence of Mineral Filler Chemical Composition on the Interaction Capacity Between Asphalt and Mineral Filler

To examine the effect of mineral filler chemical composition on the interaction capacity between asphalt and mineral filler, asphalt mastics prepared with three different types of mineral fillers, each exhibiting distinct chemical compositions, were evaluated using 90# base asphalt under identical experimental conditions of temperature and loading frequency. For analytical consistency, the chemical composition of the mineral fillers was characterized by their SiO₂ content.

Mineral filler acidity/alkalinity is a chemical property, whereas the thickness of the adsorbed asphalt film in asphalt mastic is also affected by the specific surface area of the mineral filler. Therefore, to exclusively analyze the relationship between the interaction capability of asphalt and mineral filler and the acidity/alkalinity of the mineral filler, this study introduced a coefficient d_s , defined as the adsorbed asphalt film thickness per unit of specific surface area. Based on the specific surface area and the fitted adsorbed asphalt film thickness of the three mineral fillers, the d_s values of the three types of asphalt mastics were determined. A higher d_s value indicates a stronger interaction capability between asphalt and mineral filler.

The relative content of chemical components in the three mineral fillers was determined via XRD analysis. The diffraction peak patterns of the XRD results were integrated, and the relative content of each compound was calculated by dividing the sum of the peak areas corresponding to each compound by the total peak area. The relative content of each compound is shown in Table 6. Figure 11 illustrates the relationship between the d_s value of asphalt mastic and the SiO₂ content of mineral filler.

Table 6. Relative content of chemical components in fillers.

Filler Type	Chemical Component (%)						
	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	MgO	CaO	Na ₂ O	K ₂ O
Limestone filler	2.13	0.53	0.25	9.48	51.48	0.11	0.2
Fly ash	45.01	16.52	17.1	1.2	2.8	0.5	0.6
Coal gangue filler	35.42	25.93	2.39	1.22	19.01	0.86	0.21

Table 6 presents the relative chemical composition of the three mineral fillers. Among them, limestone filler has the lowest SiO₂ content and the highest CaO content (51.48%), being an alkaline compound. Coal gangue filler contains 19.01% CaO and a relatively high amount of Al₂O₃ (25.93%)—another alkaline component. In contrast, fly ash has a lower content of alkaline compounds and primarily consists of acidic SiO₂.

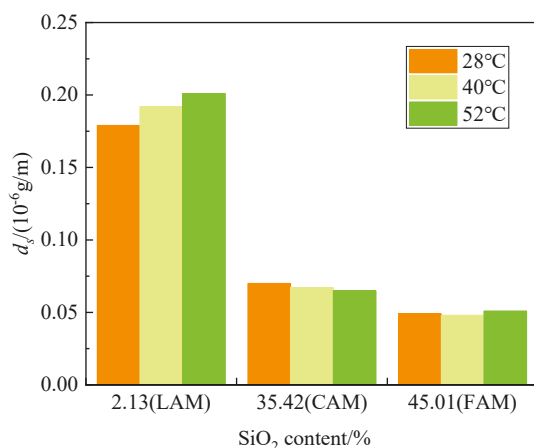


Figure 11. Effect of SiO_2 content on the thickness of the adsorbed asphalt film.

As shown in Figure 11, within the critical volume fraction range of mineral filler, the interaction capability between asphalt and mineral filler decreases with increasing SiO_2 content. This is because asphalt contains a significant amount of asphaltenes and anhydrides, while SiO_2 , an acidic component in mineral filler, negatively impacts the adhesion between asphalt and mineral filler [41]. A higher SiO_2 content makes it more difficult for asphalt to interact physically and chemically with the mineral filler, resulting in a thinner adsorbed asphalt film and weaker interaction capability. According to chemical reaction theory, when the content of alkaline components (e.g., CaO) in mineral filler increases, the physical and chemical interactions with asphalt are enhanced, leading to a thicker adsorbed asphalt film and stronger interaction capability.

4.4.2. Influence of Temperature on the Interaction Capability Between Asphalt and Mineral Fillers

The effect of temperature on the interaction capability between asphalt and mineral fillers was examined by analyzing the variation in adsorbed asphalt film thickness for three types of asphalt mastics, as illustrated in Figure 12.

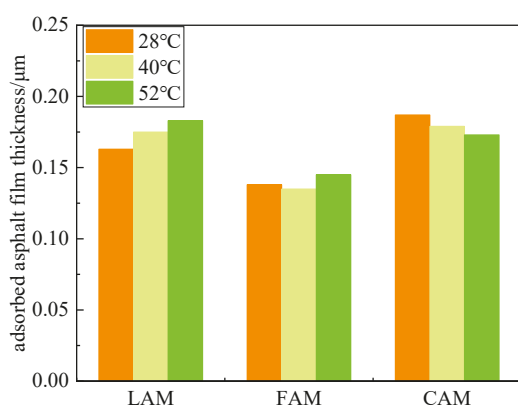


Figure 12. Variation in adsorbed asphalt film thickness with temperature for different asphalt mastics.

Figure 12 demonstrates that, within the investigated range of mineral filler volume fractions, the adsorbed asphalt film thickness of limestone asphalt mastic increases with rising temperature. This trend can be attributed to two factors. The primary reason is that when the temperature surpasses the softening point of asphalt (48°C), asphalt transitions from a semi-solid to a viscous flow state, improving its contact and wetting ability with the mineral filler [42]. Additionally, elevated temperatures enhance the adhesion between asphalt and the alkaline mineral components of the filler. Since limestone contains 51.48%

CaO, the adhesion between asphalt and CaO strengthens with increasing temperature, thereby improving the interaction between asphalt and the limestone mineral filler.

For fly ash asphalt mastic, the adsorbed asphalt film thickness exhibits minimal variation with temperature. This is primarily due to the low CaO content in fly ash, which is only 2.8%. As a result, the adhesion between asphalt and CaO remains nearly unchanged with rising temperature. Fly ash particles possess a dense CaO-SiO₂-Al₂O₃ system, and their glassy phase forms a smooth, compact, and robust Si-O-Si and Si-O-Al network structure [43]. Therefore, even when the temperature exceeds the asphalt softening point and transitions it into a viscous flow state, the effective wetting of fly ash particles remains limited. Consequently, temperature variations exert little influence on the interaction between asphalt and fly ash.

In coal gangue asphalt mastic, the adsorbed asphalt film thickness decreases as temperature rises. This reduction is primarily due to the high SiO₂ content and large specific surface area of coal gangue. With increasing temperature, asphalt more effectively contacts coal gangue; however, the acidic SiO₂ inhibits its interaction with the mineral filler. Even beyond the asphalt softening point, the inhibitory effect of SiO₂ in coal gangue surpasses the enhancement from asphalt wetting. As a result, the interaction capability continues to weaken at 52 °C.

4.4.3. Influence of Frequency on the Interaction Capability Between Asphalt and Mineral Filler

Figure 13 illustrates the variation in adsorbed asphalt film thickness with experimental frequency for three types of asphalt mastics at different temperatures. As the influence of the filler-to-binder ratio is not considered in this analysis, the film thickness results at a constant filler-to-binder ratio were selected for investigation. Given that the mineral filler volume fraction of FAM, CAM, and LAM falls within the critical range at a filler-to-binder ratio of 0.4, this value was chosen to minimize the influence of inter-particle interactions on the analysis results.

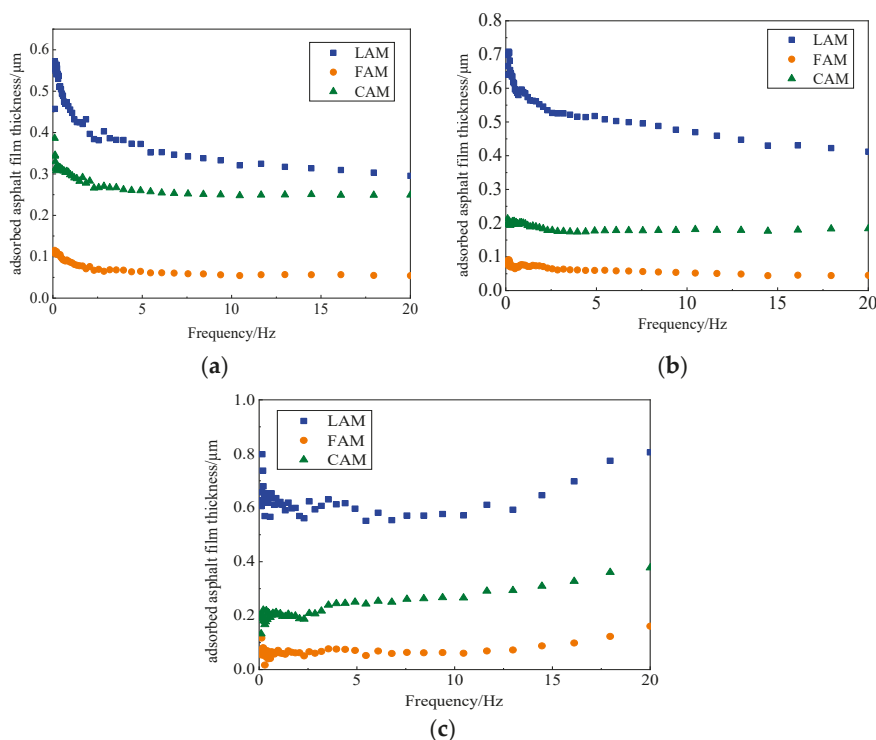


Figure 13. Variation in adsorbed asphalt film thickness with experimental frequency at different temperatures: (a) 28 °C; (b) 40 °C; (c) 52 °C.

Figure 13 demonstrates that adsorbed asphalt film thickness varies with experimental frequency, reflecting changes in interaction capability. Thus, the interaction capability between asphalt and mineral filler is frequency-dependent. Figure 13a,b indicate that at 28 °C and 40 °C, adsorbed asphalt film thickness decreases as frequency increases. This suggests that at lower temperatures, increasing frequency weakens the interaction capability between asphalt and mineral filler. As frequency increases, the shear modulus of asphalt and asphalt mastic rises, causing asphalt to gradually transition from viscoelastic to elastic behavior [44]. This transition reduces the wetting ability of asphalt, thereby weakening its interaction with the mineral filler. Figure 13c reveals that at 52 °C, adsorbed asphalt film thickness increases with frequency. Since this temperature surpasses the asphalt softening point (48 °C), the material transitions from a semi-solid to a viscous flow state. At this stage, the wetting effect of asphalt outweighs the influence of frequency, enhancing its interaction with the mineral filler.

5. Conclusions

This study employed the Hashin model, Mori–Tanaka model, and generalized self-consistent (GSC) model to calculate the adsorbed asphalt film thickness, compare and optimize these calculations, and establish this factor as an indicator for evaluating interaction capability. Additionally, the effects of temperature, frequency, and the chemical composition of mineral fillers on the rheological properties of asphalt mastics and their interaction capability were examined. The primary conclusions are as follows:

- (1) The adsorbed asphalt film thickness was calculated using the Hashin model, the Mori–Tanaka model, and the GSC model. The resulting thickness ranged from 0.01 to 0.37 μm , determined by calculating the difference between the effective and initial volume fractions of the mineral filler.
- (2) A comparison was made of the adsorbed asphalt film thicknesses, calculated using the three micromechanical models. The film thickness based on the Hashin model was slightly greater than that derived from the Mori–Tanaka model, while the GSC model produced a significantly thinner film. In comparison to the film thicknesses obtained from the GSC and Mori–Tanaka models, the Hashin model provided a clearer physical interpretation, with no negative values and a simpler formula. It offered a more effective evaluation of the interaction capability between asphalt and mineral filler.
- (3) The shear modulus ratios of asphalt mastic under different filler volume fractions were subjected to nonlinear fitting, which allowed for a more accurate calculation of the critical filler volume fractions. The critical filler volume fractions for LAM, FAM, and CAM were determined to be 30.65%, 44.96%, and 41.15%, respectively. These values provide a basis for the scope of research on the interaction between asphalt and fillers. The results of the fitting provided a single film thickness for each type of asphalt mastic, demonstrating good fitting quality. At all temperatures, the film thickness followed a consistent trend: CAM > LAM > FAM. This method eliminated the dependence of the interaction evaluation indicator on the mineral filler volume fraction.
- (4) The interaction capacity between asphalt and mineral fillers was systematically investigated. It was found that the incorporation of mineral fillers significantly enhances the complex shear modulus of base asphalt, thereby increasing its elasticity while reducing its viscosity. Within the critical filler volume fraction range, rutting resistance is predominantly influenced by the material properties of the fillers. The interaction capability between asphalt and mineral filler is influenced by multiple factors. A higher SiO_2 content in mineral filler weakens the interaction with asphalt. The effect

of frequency varies with temperature: below the softening point, a higher frequency reduces the interaction capability, whereas above this threshold, the interaction capability increases. The interaction between limestone filler and asphalt strengthens as temperature rises. The compact $\text{CaO-SiO}_2\text{-Al}_2\text{O}_3$ structure of fly ash minimizes sensitivity to temperature changes, while the high SiO_2 content in coal gangue inhibits its interaction with asphalt.

Author Contributions: Conceptualization, S.W. and Z.W.; methodology, S.W. and Z.W.; validation, S.W., Z.W. and H.Y.; formal analysis, S.W. and A.Y.; investigation, S.W. and Z.W.; resources, Z.W., F.Z. and Z.H.; data curation, S.W., A.Y. and X.M.; writing—original draft preparation, S.W. and A.Y.; writing—review and editing, S.W. and Z.W.; visualization, S.W. and A.Y.; supervision, Z.W. and Z.G.; project administration, Z.W. and D.G.; funding acquisition, Z.W. and S.W. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by Heilongjiang Provincial Natural Science Foundation of China [grant no YQ2022E038], the Key Project of Scientific Research Project of Hunan Provincial Department of Education (Grant No. 24A0752), the Excellent Youth Project of Scientific Research Project of Hunan Provincial Department of Education (Grant No. 24B1069), the National Natural Science Foundation of China (Grant No. 52278454).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: All relevant data are contained in the present manuscript.

Acknowledgments: The authors would like to thank Jianming Liu from Shiyanjia Lab (www.shiyanjia.com (accessed on 1 March 2025)) for the BET analysis.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Sousa, M.; Dinis Almeida, M.; Fael, C.; Bentes, I. Permeable Asphalt Pavements (PAP): Benefits, Clogging Factors and Methods for Evaluation and Maintenance—A Review. *Materials* **2024**, *17*, 6063. [CrossRef] [PubMed]
2. Habbouche, J.; Hajj, E.Y.; Piratheepan, M.; Sebaaly, P.E. Impact of high polymer modification on reflective cracking performance life of asphalt concrete overlays. *Int. J. Pavement Res. Technol.* **2020**, *13*, 510–523. [CrossRef]
3. Liang, J.; Zhang, Q.; Gu, X. Classification of Asphalt Pavement Defects for Sustainable Road Development Using a Novel Hybrid Technology Based on Clustering Deep Features. *Sustainability* **2024**, *16*, 10145. [CrossRef]
4. Wang, Z.; Guo, N.; Wang, S.; Xu, Y. Prediction of highway asphalt pavement performance based on Markov chain and artificial neural network approach. *J. Supercomput.* **2021**, *77*, 1354–1376. [CrossRef]
5. Veytskin, Y.; Bobko, C.; Castorena, C.; Kim, Y. Nanoindentation investigation of asphalt binder and mastic cohesion. *Constr. Build. Mater.* **2015**, *100*, 163–171. [CrossRef]
6. Das, A.K.; Singh, D. Investigation of rutting, fracture and thermal cracking behavior of asphalt mastic containing basalt and hydrated lime fillers. *Constr. Build. Mater.* **2017**, *141*, 442–452. [CrossRef]
7. Zeng, M.; Wu, C. Effects of Type and Content of Mineral Filler on Viscosity of Asphalt Mastic and Mixing and Compaction Temperatures of Asphalt Mixture. *Transp. Res. Rec.* **2008**, *2051*, 31–40. [CrossRef]
8. Mansor, N.; Sevil, K. Effect of filler types and asphalt penetration grade on properties of asphalt mastic. *Petrol. Sci. Technol.* **2018**, *36*, 1503–1509. [CrossRef]
9. Pereira, L.; Freire, A.C.; da Costa, M.S.; Antunes, V.; Quaresma, L.; Micaelo, R. Experimental study of the effect of filler on the ductility of filler-bitumen mastics. *Constr. Build. Mater.* **2018**, *189*, 1045–1053. [CrossRef]
10. Ibarra, L.; Paños, D. Dynamic properties of thermoplastic butadiene–styrene (SBS) and oxidized short carbon fiber composite materials. *J. Appl. Polym. Sci.* **1998**, *67*, 1819–1826. [CrossRef]
11. Ziegel, K.D.; Romanov, A. Modulus reinforcement in elastomer composites. I. Inorganic fillers. *Appl. Polym. Sci.* **2010**, *17*, 1041–1050. [CrossRef]
12. Einstein, A. Eine neue Bestimmung der Moleküldimensionen. *Ann. Phys.* **1906**, *324*, 289–306. [CrossRef]
13. Chong, J.S.; Christiansen, E.B.; Baer, A.D. Rheology of Concentrated Suspensions. *J. Appl. Polym. Sci.* **1971**, *15*, 2007–2021. [CrossRef]

14. Tan, Y.; Li, X.; Wu, J. Evaluation Indices of Interaction Ability of Asphalt and Aggregate Based on Rheological Characteristics. *J. Wuhan Univ. Technol.* **2012**, *27*, 979–985. [CrossRef]
15. Tan, Y.Q.; Wu, J.T.; Li, X.M.; Ji, L. Evaluation indices of interaction ability of asphalt and aggregate. *J. Harbin Inst. Technol.* **2009**, *41*, 81–84. [CrossRef]
16. Liu, G.; Jia, Y.; Pan, Y.; Yang, T.; Zhao, Y.; Zhang, J. Quantitative comparison of evaluation indices for asphalt–filler interaction ability within filler critical volume fraction. *Road Mater. Pavement Des.* **2018**, *21*, 906–926. [CrossRef]
17. Guo, M. Mechanism of Asphalt-Mineral Interaction and Multi-Scale Evaluation Methods. Ph.D. Thesis, Harbin Institute of Technology, Harbin, China, 2016. [CrossRef]
18. EN 13179-2:2002; Bituminous Binders—Test Methods—Part 2: Determination of the Interaction Between Bitumen and Mineral Fillers. European Committee for Standardization (CEN): Brussels, Belgium, 2002.
19. Teixeira, A.; Lesueur, D. Impact of Different Fillers on the Properties of Asphalt Mixtures. In *RILEM International Symposium on Bituminous Materials*; ISBM 2020, RILEM Bookseries; Springer: Cham, Switzerland, 2022; Volume 27, pp. 1007–1013. [CrossRef]
20. Tan, Y.; Guo, M. Interfacial thickness and interaction between asphalt and mineral fillers. *Mater. Struct.* **2014**, *47*, 605–614. [CrossRef]
21. Pipintakos, G.; Soenen, H.; Goderis, B.; Blom, J.; Lu, X. Crystallinity of Bitumen via WAXD and DSC and Its Effect on the Surface Microstructure. *Crystals*. **2022**, *12*, 755. [CrossRef]
22. Guo, M.; Bhasin, A.; Tan, Y.Q. Effect of mineral fillers adsorption on rheological and chemical properties of asphalt binder. *Constr. Build. Mater.* **2017**, *141*, 152–159. [CrossRef]
23. Guo, M.; Tan, Y.; Yu, J.; Hou, Y.; Wang, L. A direct characterization of interfacial interaction between asphalt binder and mineral fillers by atomic force microscopy. *Mater. Struct.* **2017**, *50*, 141. [CrossRef]
24. Wang, Z.; Guo, N.; Xu, Y.; Wang, S. Micromechanical prediction model of viscoelastic properties for asphalt mastic based on morphologically representative pattern approach. *Adv. Mater. Sci. Eng.* **2020**, *5*, 7915140. [CrossRef]
25. Wang, Z.; Jin, X.; Sun, Y.; Wang, S. Micromechanics approach to calculate the adsorbed asphalt film thickness on mineral fillers and evaluate physiochemical interactions. *Int. J. Pavement Eng.* **2024**, *25*, 2303670. [CrossRef]
26. Hashin, Z. The Elastic Moduli of Heterogeneous Materials. *J. Appl. Mech.* **1962**, *29*, 2938–2945. [CrossRef]
27. Benveniste, Y. A new approach to the application of Mori-Tanaka’s theory in composite materials. *Mech. Mater.* **1987**, *6*, 147–157. [CrossRef]
28. Christensen, R.M.; Lo, K.H. Solutions for effective shear properties in three phase sphere and cylinder models. *J. Mech. Phys. Solids* **1979**, *27*, 315–330. [CrossRef]
29. Kim, M. Development of differential scheme micromechanics modeling framework for predictions of hot-mix asphalt (HMA) complex modulus and experimental validations. *Diss. Theses-Gradworks* **2009**, *7*, 64–105. [CrossRef]
30. Yin, H.M.; Sun, L.Z.; Paulino, G.H. Micromechanics-based elastic model for functionally graded materials with particle interactions. *Acta Mater.* **2004**, *52*, 3535–3543. [CrossRef]
31. Shashidhar, N.; Shenoy, A. On using micromechanical models to describe dynamic mechanical behavior of asphalt mastics. *Mech. Mater.* **2002**, *34*, 657–669. [CrossRef]
32. Mori, T.; Tanaka, K. Average stress in matrix and average elastic energy of materials with misfitting inclusions. *Acta Metall.* **1973**, *21*, 571–574. [CrossRef]
33. Buttlar, W.; Bozkurt, D.; Al-Khateeb, G.; Waldhoff, A. Understanding Asphalt Mastic Behavior Through Micromechanics. *Transp. Res. Rec.* **1999**, *1681*, 157–169. [CrossRef]
34. Kim, Y.R.; Little, D.N. Linear Viscoelastic Analysis of Asphalt Mastics. *J. Mater. Civ. Eng.* **2004**, *16*, 122–132. [CrossRef]
35. Underwood, B.S.; Kim, Y.R. A four phase micro-mechanical model for asphalt mastic modulus. *Mech. Mater.* **2014**, *75*, 13–33. [CrossRef]
36. Li, F.; Yang, Y. Understanding the temperature and loading frequency effects on physicochemical interaction ability between mineral filler and asphalt binder using molecular dynamic simulation and rheological experiments. *Constr. Build. Mater.* **2020**, *244*, 118311. [CrossRef]
37. Li, F.; Yang, Y.; Wang, L. Evaluation of physicochemical interaction between asphalt binder and mineral filler through interfacial adsorbed film thickness. *Constr. Build. Mater.* **2020**, *252*, 119135. [CrossRef]
38. Di Benedetto, H.; Olard, F.; Sauzéat, C.; Delaporte, B. Linear viscoelastic behavior of bituminous materials: From binders to mixes. *Road Mater. Pavement Des.* **2004**, *5*, 163–202. [CrossRef]
39. Faheem, A.F.; Bahia, H.U. Modelling of Asphalt Mastic in Terms of Filler-Bitumen Interaction. *Road Mater. Pavement Des.* **2010**, *11* (Suppl. S1), 281–303. [CrossRef]
40. Fan, L.; Lin, J.T.; Li, Y.Z. Characterization of mineral filler particles related to asphalt mastic properties. *J. Wuhan Univ. Technol. (Transp. Sci. Eng.)* **2021**, *45*, 82–87.
41. Cheng, Y.; Tao, J.; Jiao, Y.; Tan, G.; Guo, Q.; Wang, S.; Ni, P. Influence of the properties of filler on high and medium temperature performances of asphalt mastic. *Constr. Build. Mater.* **2016**, *118*, 268–287. [CrossRef]

42. Tan, Y.Q.; Li, X.L.; Wu, J.T.; Luo, D.S.; Cao, L.P. Effect of temperature and loading frequency on asphalt-aggregate interaction ability. *China J. Highw. Transp.* **2012**, *25*, 65–72. [CrossRef]
43. Ke, G.J.; Yang, X.F.; Peng, H.; Jia, F.; Yue, H.T. Research progress on the mechanism of chemical activation of fly ash. *J. China Coal Soc.* **2005**, *30*, 366–370. [CrossRef]
44. Zhou, Z.X. Study on the Influence of Filler on the High and Low Temperature Performance of Asphalt Mastic and Prediction of Asphalt Mastic Properties. Master's Thesis, Hunan University, Changsha, China, 2013. [CrossRef]

Disclaimer/Publisher's Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Experimental Study on Deformation and Damage Evolution of Cracked Red Sandstone Under Freeze–Thaw Cycles

Xize Han ¹, Guangchen Sun ^{1,*}, Helin Fu ², Chao Tan ¹, Zailong Huang ¹, Peng Yin ¹, Qishu Zhang ¹, Wenchen Fan ¹ and Shuiping Yin ^{1,3}

¹ School of Civil Engineering, Central South University of Forestry and Technology, Changsha 410004, China; jshxz122@163.com (X.H.); 19973974610@163.com (C.T.); hzl4936@163.com (Z.H.); yinpeng99@sohu.com (P.Y.); zhangqishu930213@163.com (Q.Z.); fanwch@csuft.edu.cn (W.F.); ysp@csuft.edu.cn (S.Y.)

² School of Civil Engineering, Central South University, Changsha 410075, China; fu.h.l@csu.edu.cn

³ Hunan Provincial Key Laboratory of Engineering Rheology, Changsha 410004, China

* Correspondence: sailor811022@163.com

Abstract: Cracked rock masses in cold regions are subjected to freeze–thaw cycles over extended periods, resulting in freeze–thaw deformation. The combined effects of freeze–thaw cycling and the depth of cracks significantly influence the stability and durability of underground rock engineering in these regions. In some cold regions with minimal annual rainfall, rock masses are unable to absorb external water during freeze–thaw cycles. As freeze–thaw deformation progresses, the rock transitions naturally from a saturated state to an unsaturated state. To investigate the deformation damage mechanisms and evolution patterns of saturated red sandstone with initial non-penetrating cracks of varying depths (20 mm, 30 mm, 40 mm) under freeze–thaw cycling conditions without external water replenishment and with naturally varying saturation levels, relevant freeze–thaw cycle experiments and strain monitoring were conducted. The results indicate that cracked red sandstone experiences residual strain in each freeze–thaw cycle, which gradually accumulates, leading to irreversible freeze–thaw damage deformation. The cumulative residual strain of the rock specimen after 45 freeze–thaw cycles was 40.69 times greater than the residual strain from the first cycle. Additionally, the freeze–thaw strain characteristic values exhibited a clear correlation with crack depth. These findings provide experimental methods and data references for analyzing the deformation and failure mechanisms of cracked rock induced by freeze–thaw damage in cold regions.

Keywords: cracked red sandstone; freeze–thaw cycles; freeze–thaw deformation; residual strain; freeze–thaw damage effects

1. Introduction

As transportation networks continue to improve, road and tunnel construction projects are gradually extending into cold regions with harsh climatic conditions [1]. These regions are widely covered by permafrost, which refers to various types of rocks and soils containing ice at or below 0 °C [2,3]. The study of frozen rock has emerged as a more specialized direction from permafrost research. Rocks, as porous media composed of solid skeleton particles and intergranular pores, have structures made up of interconnected solid skeleton particles, with the pores typically filled by one or more fluids [4–6]. In cold regions, temperatures often drop below zero, and seasonal changes can cause drastic temperature fluctuations, subjecting rocks to freeze–thaw cycles [7–9]. Under cyclic influences of temperature stress and pore ice heave forces, rock skeleton particles experience uneven expansion and contraction, which expands the internal pores, resulting in damage to the macroscopic structure and a decline in mechanical properties [10–12]. Freeze–thaw deformation disrupts the original equilibrium state of the rock mass, affecting the stability and durability of underground rock engineering in cold regions [13–15].

Many scholars have conducted extensive studies on rock deformation under freeze–thaw cycles. Tian et al. [16] conducted freeze–thaw cycle experiments on cracked red sandstone and found that under freeze–thaw conditions, the cracked red sandstone exhibited frost heave deformation. Jia et al. [17] studied the deformation patterns of rocks with different crack lengths, widths, and lithologies over time and temperature during freeze–thaw cycles, finding that deformation damage was mainly due to cumulative effects from temperature damage, frost heave, and thermal expansion. Kang et al. [18] investigated the deformation characteristics of rocks in low-temperature environments and simulated frost heave deformation in cold-region tunnels. By testing strain characteristics in dry and water-saturated rock specimens under low temperatures, they found that water-saturated specimens exhibited residual strain during contraction, frost heave, thawing, and thermal expansion stages, while dry specimens only displayed linear elasticity. Jia et al. [19] discovered that specimens with 50% and 90% saturation exhibited significant deformation during freeze–thaw cycles. Pei et al. [20] conducted experiments on the strain process of cracked rocks with different lithologies under freeze–thaw cycles, finding differences between saturated and dry states, with freeze–thaw cycle frequency affecting strain values. Xia et al. [21] conducted unidirectional freezing experiments on saturated cracked rock, demonstrating that the frost heave effect caused by cracks differed significantly from that of the rock matrix, with cracked rock masses displaying strong heterogeneous frost heave behavior. Huang et al. [22] developed a new elasto-plastic model to estimate the long-term frost heave and deformation of porous rocks in cold regions under freeze–thaw conditions, considering factors such as pore size, matrix yield stress, and water saturation. Lv et al. [23] proposed a thermo-mechanical coupled elasto-plastic model for porous rocks under freeze–thaw conditions, which can accurately predict the temperature changes and freeze–thaw deformation processes of porous rocks. Most scholars use strain gauge methods for monitoring deformation, as exemplified by Wang et al. [24], who studied the principle of measuring linear expansion coefficients using strain gauges and compared this method with traditional quartz tube dilatometers. Their results showed that strain gauge monitoring is simple, reliable, and accurate.

In summary, numerous scholars have conducted extensive experimental and theoretical studies on the freeze–thaw deformation of cracked rocks, primarily focusing on variables such as crack length, crack width, lithology, and rock saturation. However, most of the existing research has concentrated on either saturated rocks or rocks with constant saturation levels. In practical engineering scenarios, the saturation of rocks during long-term freeze–thaw cycles is not constant. For example, in some cold regions north of the Tropic of Cancer, the autumn and winter climates are characterized by cold, dry conditions, significant diurnal temperature variations, and low annual precipitation [25–27]. During freeze–thaw cycles, rocks cannot adequately replenish or absorb external moisture, resulting in the natural transition of rock masses from a saturated state to an unsaturated state as freeze–thaw deformation progresses. Furthermore, the crack planes in cracked engineering rock masses serve as weak failure surfaces, making the rock highly susceptible to failure along these planes under external loads. Additionally, the freeze–thaw deformation of rocks containing cracks has a more severe impact on the stress equilibrium of rock masses, significantly affecting their stability [28–30]. However, the current research predominantly focuses on the freeze–thaw deformation of rocks with through-going cracks, with crack length and width being the primary research variables. Studies on rocks containing initial non-penetrating cracks of varying depths remain scarce. The strain characteristic values of rocks during freeze–thaw cycles may exhibit significant differences with increasing crack depth.

Therefore, the research on the deformation characteristics of saturated rocks with initial non-penetrating cracks of varying depths under long-term freeze–thaw cycles, where external water cannot be replenished and saturation changes naturally, remains limited but highly significant. Based on these considerations, this study simulates realistic freeze–thaw cycle conditions to investigate the deformation evolution and damage mechanisms of

red sandstone specimens containing initial non-penetrating cracks with depths of 20 mm, 30 mm, and 40 mm under freeze–thaw cycles without external water supplementation. Freeze–thaw cycle experiments and strain monitoring were conducted on red sandstone specimens with different crack depths under isolated freeze–thaw conditions. The deformation behavior of red sandstone with varying crack depths was examined over time and temperature during freeze–thaw cycles. The evolution of freeze–thaw strain characteristic values were also analyzed, alongside the freeze–thaw deformation evolution mechanisms of cracked red sandstone. The experimental results provide valuable data references for understanding the deformation and failure mechanisms of cracked rocks subjected to freeze–thaw damage effects in cold regions. Additionally, the naming conventions for the symbols used in this paper are shown in Appendix A Table A1.

2. Materials and Methods

2.1. Materials

Since red sandstone is widely found in practical engineering projects such as tunnels and foundation pits, and it is prone to disintegration and fragmentation under environmental influences after excavation, its strength significantly decreases when soaked in water. Additionally, red sandstone's physical and mechanical properties are easily affected by freeze–thaw cycles and wet–dry cycles. Therefore, red sandstone was selected as the test material for this study. The red sandstone specimens used in this experiment were cored from the original rock and then cut and polished to produce standard cylindrical specimens with dimensions of $\Phi 50 \text{ mm} \times 100 \text{ mm}$. All specimens were taken from the same rock block to ensure experimental accuracy and comparability, effectively reducing errors caused by specimen variability.

Considering that open-type cracks significantly impact the stability of underground rock engineering, this study investigated the influence of crack depth on rock deformation. A minimally invasive water jet cutting method was employed to prefabricate non-penetrating open-type cracks in the processed standard rock specimens. On the radial surface of the $\Phi 50 \times 100 \text{ mm}$ cylindrical rock specimens, a vertical water jet cut was made from the midpoint of the 100 mm height downward, creating a non-penetrating crack with a length of 30 mm, a width of 3 mm, and an inclination angle of 45° . The short edge of the prefabricated crack was designated as the reference point, from which the penetration depth was measured inward. Three different crack depths—20 mm, 30 mm, and 40 mm—were prefabricated, ensuring that the cracks did not fully penetrate the rock specimens. Three specimens of cracked rock for each depth were prepared and divided into three groups for testing to ensure reproducibility. Each group of rock specimens consists of one specimen with a 20 mm crack depth, one with a 30 mm crack depth, and one with a 40 mm crack depth. The dimensions and actual specimens of the rock samples are shown in Figure 1a,b, respectively. The basic physical property parameters of the rock specimens are shown in Table 1. The standard deviation and covariance of these parameters are presented in Table A2. To ensure the reproducibility and clarity of this study, XRD mineral composition analysis was conducted on the red sandstone used in the experiments, revealing the mineral composition of the rock specimens [31,32]. The mineral composition was tested using a Rigaku SmartLab X-ray diffractometer manufactured by Rigaku Corporation, Tokyo, Japan. The step size was 0.02° , with a copper target wavelength of $1.5406 \text{ \AA}$. The scanning range was 5° to 80° , and the scanning speed was $10^\circ/\text{min}$. The measured mineral composition consists of eight minerals: quartz, Albite, Hematite, Calcite, Microcline, Anorthite, Muscovite, and Montmorillonite. The primary elements identified include Fe, Ca, K, Si, Al, Na, O, C, and H. The XRD diffraction pattern of the rock specimen is presented in Figure 1c, and the mineral composition proportions of the rock specimen are shown in Figure 1d.

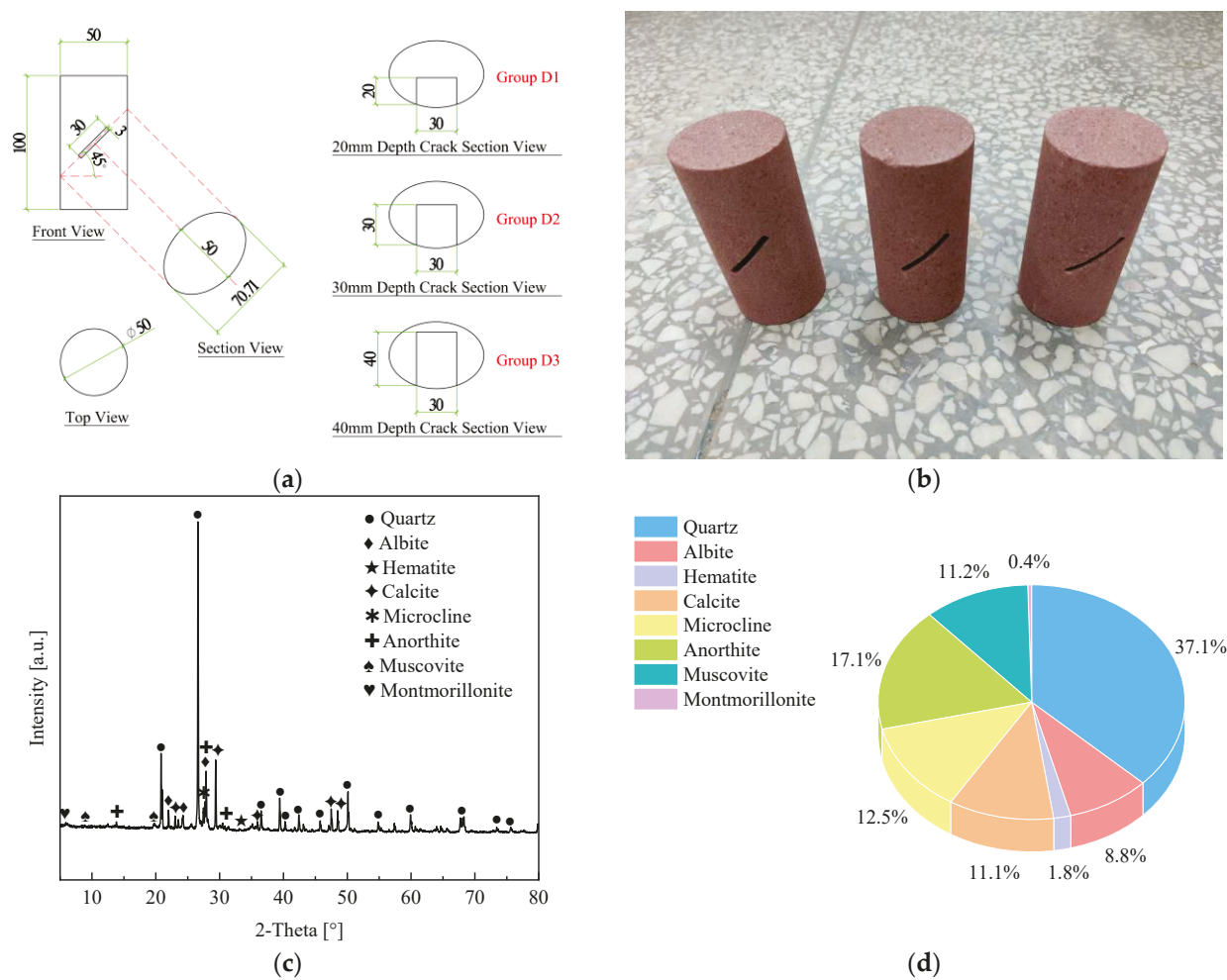


Figure 1. Rock specimen: (a) rock specimen dimensions; (b) actual rock specimen; (c) the XRD diffraction pattern of the rock specimen; (d) the mineral composition proportions of the rock specimen.

Table 1. Basic physical property parameters of the rock specimen.

Category	Crack Length [mm]	Crack Width [mm]	Crack Depth [mm]	Crack Volume * [mm ³]	Dry Density [g/cm ³]	Saturated Density [g/cm ³]	Water Absorption [%]	Porosity [%]
D1	30	3	20	1943	2.085	2.215	6.222	13.972
D2	30	3	30	2843	2.079	2.208	6.193	14.338
D3	30	3	40	3743	2.073	2.201	6.142	14.660

* The crack volume is considered as a part of the total volume of the rock specimen.

2.2. Strain Monitoring Method

Most natural rock masses contain initial cracks, and under external loading, the localized area near the crack tip is highly prone to damage or failure. The stress–weak plane of a cracked rock mass is directly related to the orientation of the crack. Therefore, monitoring strain changes around the crack allows for investigating its deformation behavior, which in turn can help assess the freeze–thaw damage in cracked rock masses. To monitor the strain changes around the crack, measurement points were selected on the rock specimen surface at the edges of the crack, in a direction perpendicular to the short edge and parallel to the long edge of the crack. Additionally, two symmetric points were selected as reference measurement points, as shown in Figure 2. Strain gauges were attached at all four measurement points to measure the strain values and monitor the variation patterns.

The signal transmitted by the strain gauge was collected using a static resistance strain gauge instrument.

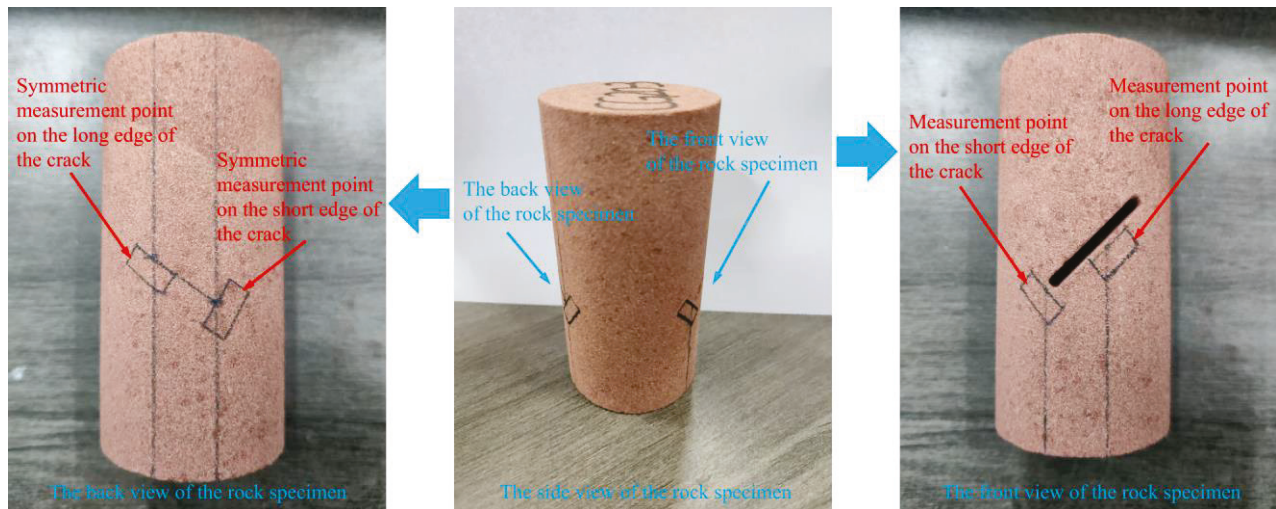


Figure 2. Arrangement of strain measurement points on the rock specimen.

Due to the significant influence of environmental temperature on the resistance of strain gauges, the resistance of the strain gauge changes with fluctuations in external temperature. The resistance change caused by temperature variations is of the same order of magnitude as that caused by the strain in the specimen, leading to considerable measurement errors, referred to as temperature errors of the strain gauge. This occurs for two reasons: first, the resistance wire of the strain gauge has a certain temperature coefficient, and second, the thermal expansion coefficients of the resistance wire and the test material are different.

Therefore, it is necessary to eliminate the temperature error of the strain gauge to obtain accurate strain values, and the bridge compensation method is a commonly used approach for this purpose. Quartz glass is typically selected as the material for the temperature compensation strain gauge due to its extremely low and stable thermal expansion coefficient, which is an order of magnitude lower than that of typical rock materials, with a thermal expansion coefficient of $5.3 \times 10^{-7} \text{ }^{\circ}\text{C}^{-1}$. The specific procedure is as follows: On the static resistance strain gauge instrument, select the half-bridge circuit configuration. Attach the working strain gauge to the measurement point on the rock specimen and attach the temperature compensation strain gauge to the quartz glass. During testing, place both the temperature compensation strain gauge and the working strain gauge in the same temperature environment, as shown in Figure 3a. Since strain gauges malfunction when exposed to water, waterproofing measures must be applied when attaching the strain gauges, as shown in Figure 3b. The actual working strain gauge and its waterproof treatment are shown in Figure 3c,d, respectively. The actual temperature compensation strain gauge and its waterproof treatment are shown in Figure 3e,f, respectively. The strain gauge model is “BFH120-5AA-D300”, with a sensitivity coefficient of $2.0 \pm 1\%$ and an accuracy grade of Class A.

Under half-bridge compensation, the readings from the static resistance strain gauge instrument do not directly represent the actual strain values of the rock specimen. After mathematical calculations, Kang et al. [18] derived the following formula to calculate the actual strain values of the rock specimen, as shown in Equation (1):

$$\varepsilon = \left(\frac{\varepsilon_{\mu}}{\Delta t} + \beta_c \right) \Delta t = \varepsilon_{\mu} + \beta_c \Delta t \quad (1)$$

where ε_{μ} is the measured reading from the static resistance strain gauge instrument; β_c is the linear thermal expansion coefficient of quartz glass; and Δt is the temperature change value.

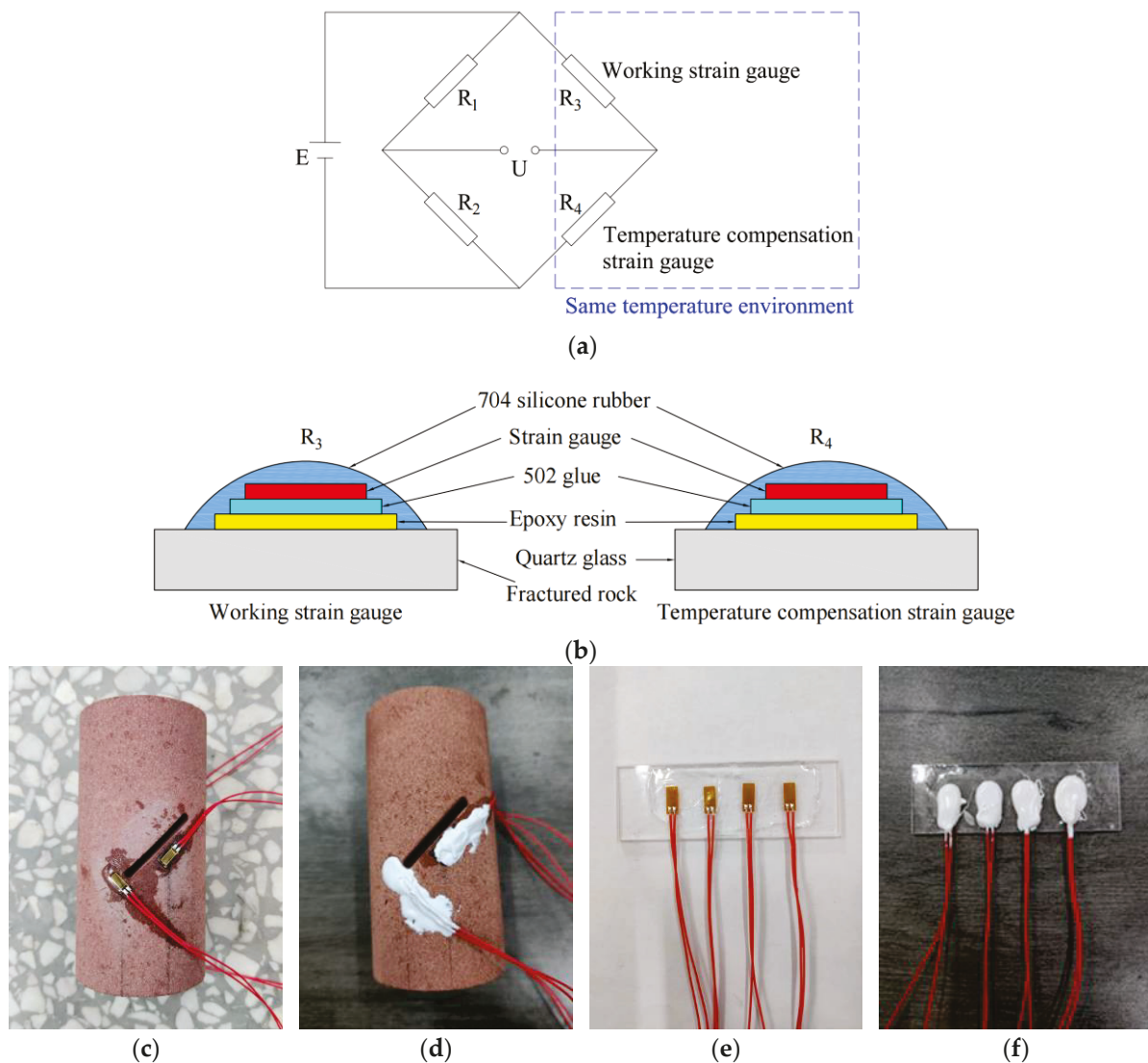


Figure 3. Bridge compensation method: (a) half-bridge circuit; (b) waterproofing measures for strain gauges; (c) actual working strain gauge; (d) physical working strain gauge with waterproof treatment; (e) actual temperature compensation strain gauge; (f) actual temperature compensation strain gauge with waterproof treatment.

2.3. Freeze–Thaw Cycle Testing Equipment and Method

2.3.1. Freeze–Thaw Cycle Testing Equipment

In this experiment, a self-designed freeze–thaw cycle system was assembled, as shown in Figure 4. Additionally, a water immersion sensor and a smoke alarm were employed as safety protection measures, while a static resistance strain gauge instrument, a temperature logger, and a computer were used for monitoring and data acquisition. The freeze–thaw cycle system primarily includes a numerical control refrigerator, a heater, a polyethylene air duct, a router, WIFI smart sockets, wooden supports, iron weights, a time relay, and an electric push rod. The numerical control refrigerator functions as the cooling device, while the heater is used for heating and ventilation. The WIFI smart socket has a timed automatic switch function, allowing users to set the on/off times via a mobile device app. Two WIFI smart sockets are used to connect the power sources for the refrigerator and the heater, enabling timed control of cooling and heating functions. An electric push rod is installed on the left side of the numerical control refrigerator to enable the opening and closing of the refrigerator door. The push rod is connected to the power supply through a time relay, and the interval for pushing and pulling the refrigerator door can be set on the control panel of

the time relay, allowing the push rod to open and close the refrigerator door at set intervals automatically. With this configuration, when the push rod closes the refrigerator door, the refrigerator power is turned on, and the heater remains off, enabling the freezing function. When the push rod opens the refrigerator door, the refrigerator power is turned off, and the heater is activated, enabling the thawing function. This cycle repeats continuously, achieving automatic freeze–thaw cycling. The freeze–thaw cycle testing system underwent multiple trial operation tests, during which no faults occurred. The freezing temperature reached $-20\text{ }^{\circ}\text{C}$ ($\pm 2\text{ }^{\circ}\text{C}$), and the melting temperature reached $20\text{ }^{\circ}\text{C}$ ($\pm 2\text{ }^{\circ}\text{C}$), meeting the requirements of the freeze–thaw cycle testing.

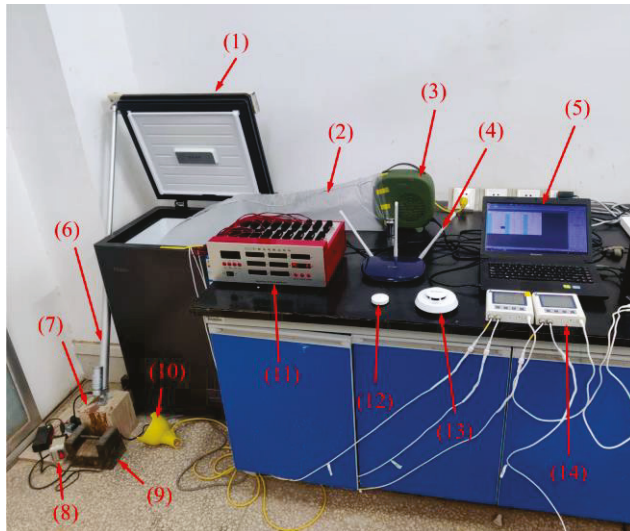


Figure 4. Freeze–thaw cycle testing system: (1) numerical control refrigerator; (2) polyethylene air duct; (3) heater; (4) router; (5) computer; (6) electric push rod; (7) wooden support; (8) time relay; (9) iron weight; (10) WIFI smart socket; (11) static resistance strain gauge instrument; (12) water immersion sensor; (13) smoke alarm; (14) temperature logger.

2.3.2. Freeze–Thaw Cycle Testing Method

There are generally two methods for saturating rock specimens: the vacuum saturation method and the free immersion method. In this experiment, the vacuum saturation method was not used because the low-pressure environment created by vacuum extraction may cause unnecessary damage to the rock specimens due to tension. Instead, the free immersion method was used to saturate the rock specimens for 48 h.

- (1) The rock specimens were placed in an electric oven and baked at $105\text{ }^{\circ}\text{C}$ for 24 h. They were then removed and cooled in a desiccator. A temperature gun was used to measure the surface temperature of the specimens until the surface temperature matched the ambient room temperature.
- (2) Using the free immersion method, the rock specimens were placed in a water tank, and water was first added to cover $1/4$ of the specimen height. Afterward, every 2 h, water was added to cover $1/2$ and then $3/4$ of the specimen height. After 6 h, the specimens were fully submerged. After the rock specimens freely absorbed water for 48 h, they were removed, and the surface water was wiped off. Each specimen was then completely wrapped in transparent polyethylene cling film and individually sealed in transparent polyethylene sealing bags to prevent moisture loss during the freeze–thaw cycle tests.
- (3) The room temperature was maintained at $20\text{ }^{\circ}\text{C}$ ($\pm 2\text{ }^{\circ}\text{C}$) using an air conditioner. The sealed rock specimens and temperature compensation strain gauges were placed flat in storage baskets inside the numerical control refrigerator, with wooden strips on both sides to prevent the specimens from rolling during the test, as shown in Figure 5. Additionally, the temperature control group specimens were placed upright in the

storage baskets to measure temperature changes during the freeze–thaw cycles. Once all the specimens were securely placed, the control software for the static resistance strain gauge was initiated, setting the data acquisition mode to “monitoring” at a frequency of 1 Hz, and the strain gauge was zeroed. Simultaneously, the freeze–thaw cycle system was started. The time relay was set to push and pull the electric push rod at intervals of 4 h, and the target freezing temperature for the numerical control refrigerator was set to $-20\text{ }^{\circ}\text{C}$. The actual operation of the freeze–thaw cycle system was as follows: initially, the numerical control refrigerator was turned on, and the door remained closed, keeping the specimens in a sealed environment at $-20\text{ }^{\circ}\text{C}$ ($\pm 2\text{ }^{\circ}\text{C}$) for 4 h. Then, the refrigerator power was automatically turned off, and the refrigerator door was automatically opened by the electric push rod. After 15 min, the heater began operating for 40 min, raising the specimen temperature to $20\text{ }^{\circ}\text{C}$ ($\pm 2\text{ }^{\circ}\text{C}$). The specimens were maintained at $20\text{ }^{\circ}\text{C}$ ($\pm 2\text{ }^{\circ}\text{C}$) for 3 h and 5 min, resulting in a total thawing time of 4 h. Subsequently, the refrigerator power was automatically turned on, and the refrigerator door was automatically closed by the electric push rod, freezing the specimens for another 4 h. This process is repeated cyclically, with each freeze–thaw cycle lasting 8 h. A total of 45 freeze–thaw cycles were conducted, during which strain data were continuously collected. This process was repeated in cycles, with each freeze–thaw cycle lasting 8 h. A total of 45 freeze–thaw cycles were conducted, during which strain and temperature data were continuously collected without interruption.

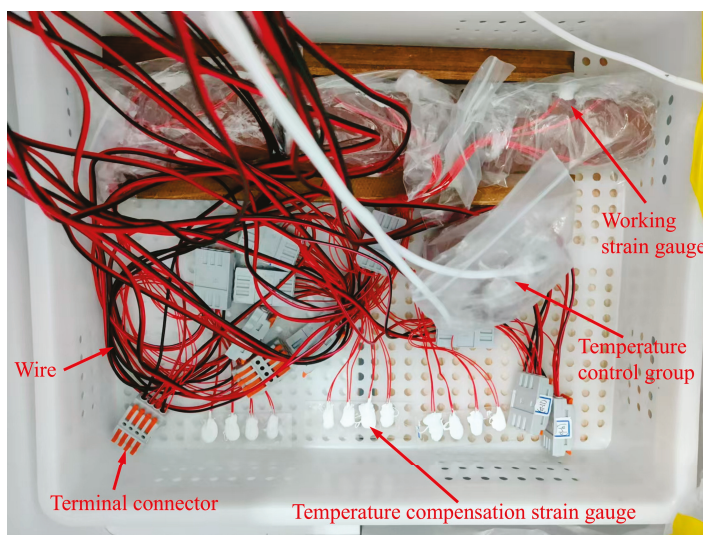


Figure 5. Freeze–thaw cycle testing.

3. Results

3.1. Strain Variation Pattern of Cracked Red Sandstone Under a Single Freeze–Thaw Cycle

The strain variation in cracked red sandstone under a single freeze–thaw cycle can be used to analyze the deformation pattern in cracked red sandstone under temperature changes. Since the strain variation trends at different points on cracked red sandstone with varying crack depths are generally similar and show no significant differences, this study selects the strain and temperature variation curve at the monitoring point on the long edge of the 20 mm deep cracked red sandstone during the tenth freeze–thaw cycle as a typical representation, as shown in Figure 6. The standard deviation and covariance of the three groups of experimental data for this measurement point are provided in Table A3.

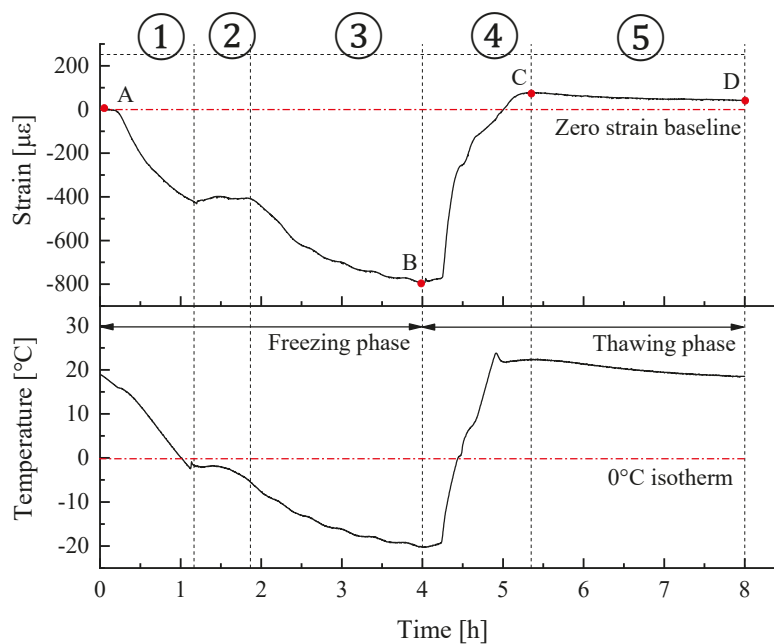


Figure 6. The strain and temperature variation curve at the monitoring point on the long edge of the 20 mm deep cracked red sandstone during the tenth freeze–thaw cycle.

During the strain collection process of cracked red sandstone in the freeze–thaw cycles, negative strain values indicate contraction deformation of the rock, while positive values represent expansion deformation. As shown in Figure 6, the strain variation trend can be described as follows: during the cooling and freezing process, the strain of cracked red sandstone gradually decreases to a minimum and then stabilizes. During the heating and thawing process, the strain gradually increases to a maximum, after which it decreases and stabilizes, resulting in residual strain. The strain variation process of cracked red sandstone in a single freeze–thaw cycle can be divided into five stages: (1) initial contraction stage; (2) stable contraction stage; (3) continuous contraction stage; (4) thermal expansion stage; and (5) stabilization stage.

- (1) **Initial Contraction Stage:** In this stage, as the surface temperature of the rock gradually decreases from 20 °C, the rock skeleton undergoes contraction deformation. The strain value starts to decrease from the initial residual strain at point A. The initial residual strain is set to 0, as shown in Figure 6.
- (2) **Stable Contraction Stage:** When the temperature of the rock decreases to around 0 °C, the rock's temperature no longer drops rapidly but stabilizes at approximately 0 °C for about 40 min. This temperature stabilization occurs because the pores within the rock contain a large amount of water, which stores heat before freezing. Since 0 °C is the freezing point of water, the water continues to release heat, keeping the temperature constant. Due to this heat release, the temperature of the rock matrix remains unchanged, and as a result, the rock skeleton ceases to contract further, causing the strain to remain stable during this stage.
- (3) **Continuous Contraction Stage:** As the temperature continues to drop, the rock skeleton undergoes further contraction deformation, and the strain value continues to decrease. When the temperature reaches the lowest point of −20 °C, the strain decreases to its minimum at point B, marking the peak of contraction deformation in the rock. During this period, the small amount of water in the rock pores undergoes a phase change, freezing into ice, which causes a slight increase in volume. The solid ice exerts pressure on the rock skeleton grains, generating a frost heave force inside the rock. However, since the rock skeleton particles in the test specimen are not strongly cemented and are relatively loosely packed, and the amount of ice in the pores is small, the expansion deformation caused by the frost heave force is minor.

The expansion caused by this frost heave force is not of the same order of magnitude as the contraction deformation induced by low temperatures, so the strain value continues to show a decreasing trend.

- (4) **Thermal Expansion Stage:** At this stage, the numerical control refrigerator stops cooling, and the door automatically opens. The rock begins to slowly warm under the influence of the ambient room temperature of 20 °C, resulting in minimal strain changes and a relatively stable state. After 15 min, the heater starts operating, causing the temperature to rise rapidly to around 20 °C. The rock skeleton undergoes expansion, and the strain increases rapidly, reaching its maximum at point C. Due to the significant temperature change on the rock surface, the rock skeleton transitions from contraction to expansion, resulting in relatively intense changes.
- (5) **Stabilization Stage:** In this stage, the heater stops running, and the rock's temperature remains consistent with the room temperature, around 20 °C. The strain slightly decreases until it stabilizes. This occurs for two reasons: On the one hand, the rapid temperature rise in the previous stage caused the rock to absorb a significant amount of heat, leading to a rock skeleton temperature slightly higher than 20 °C. This temperature difference resulted in more intense deformation. As the rock gradually cooled to stabilize at 20 °C under the influence of room temperature, the strain also decreased and stabilized. On the other hand, a small amount of water in the rock pores froze during the previous stage, transitioning from water to ice, which slightly increased its volume and applied mild pressure on the skeleton. Once the ice melted back into the water, its volume decreased, relieving the pressure on the skeleton, which led to contraction deformation. In summary, during this stage, the strain gradually decreases and stabilizes over time. After the thawing phase, the strain value at point D is slightly greater than at the initial point A, indicating that the rock skeleton experienced residual strain after one freeze–thaw cycle. This residual strain manifests as expansion deformation and is a direct indication of freeze–thaw damage in the rock.

3.2. Strain Variation Pattern in Cracked Red Sandstone Under Multiple Freeze–Thaw Cycles

Long-term, repeated freeze–thaw cycles are of great significance in studying freeze–thaw damage to rocks, as they help reveal the damage patterns in rocks in cold regions. After continuous freeze–thaw cycles, the strain variation trends at different points on cracked red sandstone with varying crack depths are generally similar, with no significant differences. In this study, the strain variation curve at the monitoring point on the long edge of the 20 mm deep cracked red sandstone during multiple freeze–thaw cycles is selected as a typical representative, as shown in Figure 7. The standard deviation and covariance of the three groups of experimental data for this measurement point are provided in Table A4. The monitoring period lasted 15 days, during which a total of 45 freeze–thaw cycles were conducted. The maximum and minimum strain values generated in cracked red sandstone during each freeze–thaw cycle are always greater than those from the previous cycle, indicating that significant expansion deformation occurs as the number of freeze–thaw cycles increases. Additionally, the final strain value in each freeze–thaw cycle does not return to the initial value but shows a slight increase, meaning that small residual strains are produced after each cycle. This demonstrates that expansion deformation occurs during every freeze–thaw cycle. The crack's long-edge monitoring point of the 20 mm depth cracked rock specimen showed an initial strain of 0 at the start of the freeze–thaw cycle. After 45 freeze–thaw cycles, the residual strain reached 529 $\mu\epsilon$, which is 40.69 times the initial residual strain of 13 $\mu\epsilon$, indicating significant expansion deformation of the crack's long edge during the freeze–thaw process. This confirms that the expansion deformation caused by freeze–thaw cycles is irreversible, indicating plastic deformation. The residual strain from each cycle accumulates, leading to gradually increasing expansion deformation, and the freeze–thaw damage intensifies as the number of cycles increases.

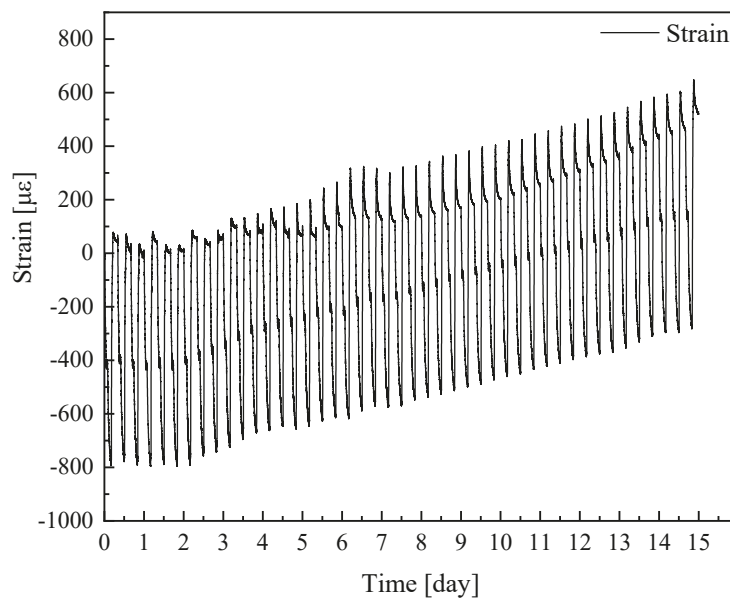


Figure 7. The strain variation curve at the monitoring point on the long edge of the 20 mm deep specimen.

3.3. Analysis of Strain Characteristic Values of Cracked Red Sandstone Under Multiple Freeze–Thaw Cycles

3.3.1. Definition of Strain Characteristic Values

The strain characteristic values of cracked rock under multiple freeze–thaw cycles mainly include cold contraction amplitude and thermal expansion amplitude. By analyzing the variation patterns in these characteristic values, it is possible to further investigate the effect of crack depth on the long-term freeze–thaw deformation characteristics of cracked rock. The definitions of cold contraction amplitude and thermal expansion amplitude are as follows.

Cold Contraction Amplitude: The absolute difference between the minimum strain value and the initial strain value during the same freeze–thaw cycle. The calculation formula is shown in Equation (2):

$$\Delta\varepsilon_1 = |\varepsilon_B - \varepsilon_A| \quad (2)$$

where ε_B is the strain value at point B in Figure 6 and ε_A is the strain value at point A in Figure 6.

Thermal Expansion Amplitude: The absolute difference between the maximum strain value and the minimum strain value during the same freeze–thaw cycle. The calculation formula is shown in Equation (3):

$$\Delta\varepsilon_2 = |\varepsilon_C - \varepsilon_B| \quad (3)$$

where ε_C is the strain value at point C in Figure 6 and ε_B is the strain value at point B in Figure 6.

3.3.2. Cold Contraction Amplitude Analysis

1. Measurement Points on the Short Edge and Long Edge of the Crack

As shown in Figure 8, with the increase in the number of freeze–thaw cycles, the cold contraction amplitudes on both the short and long edges of cracks in the rock specimens with three different crack depths remained relatively stable. The average cold contraction amplitudes for the short edge of cracks with depths of 20 mm, 30 mm, and 40 mm were 854 $\mu\varepsilon$, 903 $\mu\varepsilon$, and 1379 $\mu\varepsilon$, respectively. The average cold contraction amplitude on the

short edge of the 40 mm deep crack was 1.61 times that of the 20 mm deep crack and 1.53 times that of the 30 mm deep crack. The average cold contraction amplitudes for the long edge of cracks with depths of 20 mm, 30 mm, and 40 mm were 798 $\mu\epsilon$, 746 $\mu\epsilon$, and 1175 $\mu\epsilon$, respectively. The average cold contraction amplitude on the long edge of the 40 mm deep crack was 1.47 times that of the 20 mm deep crack and 1.58 times that of the 30 mm deep crack. It is clear that the difference in the average cold contraction amplitude on the short edge between the 20 mm and 30 mm deep cracks is relatively small, while the average cold contraction amplitude on the short edge of the 40 mm deep crack is significantly larger than the other two. The same trend is observed for the long edge of the cracks. This phenomenon may be due to the contraction deformation of rock skeleton particles under cold conditions during the freeze–thaw cycle. Since the pre-existing crack lacks part of the rock matrix internally, the surface points of the pre-existing crack have a slightly higher degree of freedom and are less constrained by the surrounding rock matrix. The greater the crack depth, the less constrained the surface points are, resulting in more pronounced deformation.

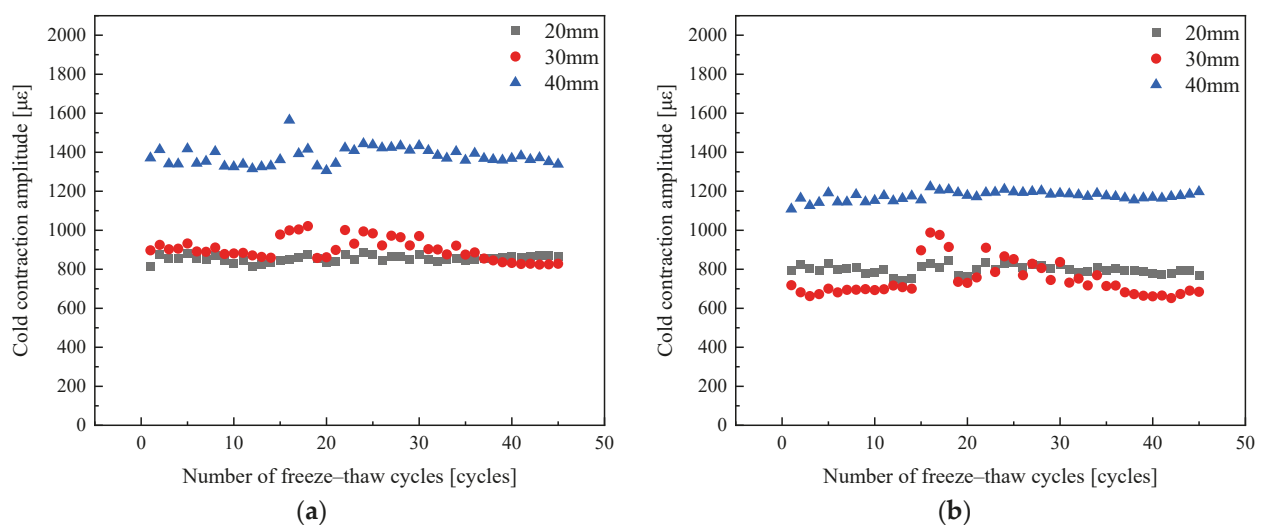


Figure 8. (a) Cold contraction amplitude of the crack’s short edge; (b) cold contraction amplitude of the crack’s long edge.

2. Symmetric Measurement Points on the Short Edge and Long Edge of the Crack

As shown in Figure 9, with the increase in the number of freeze–thaw cycles, the cold contraction amplitudes at the symmetric points on the short edge and long edge of the cracks in the rock specimens with three different crack depths remain largely unchanged. The average cold contraction amplitudes at the symmetric points on the short edge for the cracks with depths of 20 mm, 30 mm, and 40 mm are 959 $\mu\epsilon$, 1067 $\mu\epsilon$, and 763 $\mu\epsilon$, respectively. The 30 mm depth cracked rock specimen shows the largest cold contraction amplitude at the short edge symmetric point, followed by the 20 mm depth specimen, with the 40 mm depth specimen having the smallest amplitude. This indicates that the variation pattern in the cold contraction amplitude at the symmetric points of non-penetrating cracks differs from that of the surface points. For the long edge symmetric points, the average cold contraction amplitudes for the 20 mm, 30 mm, and 40 mm crack depth specimens are 983 $\mu\epsilon$, 688 $\mu\epsilon$, and 1120 $\mu\epsilon$, respectively. The 40 mm depth cracked rock specimen has the largest cold contraction amplitude at the long edge symmetric point, followed by the 20 mm depth specimen, while the 30 mm depth specimen has the smallest amplitude. When examining the relationship between the average cold contraction amplitude and crack depth, the long-edge symmetric points display a reverse correlation compared to the short-edge symmetric points.

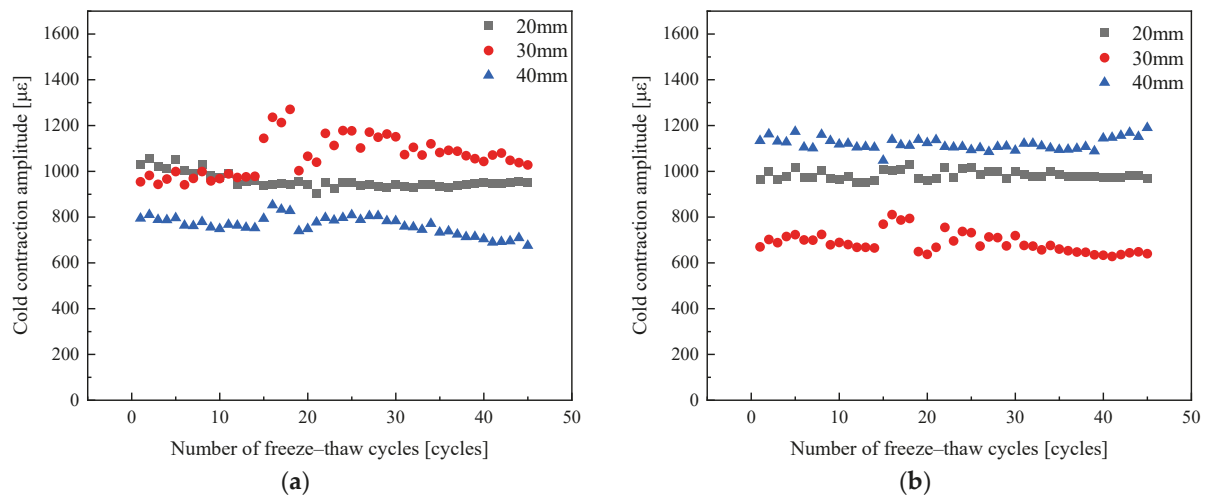


Figure 9. (a) Cold contraction amplitude at the symmetric point of the crack's short edge; (b) cold contraction amplitude at the symmetric point of the crack's long edge.

3.3.3. Thermal Expansion Amplitude

1. Measurement Points on the Short Edge and Long Edge of the Crack

As shown in Figure 10, with the increase in the number of freeze-thaw cycles, the thermal expansion amplitudes on both the short and long sides of the cracks in the rock specimens with three different crack depths remained relatively stable. The average thermal expansion amplitudes for the short edge of the cracks with depths of 20 mm, 30 mm, and 40 mm were 937 $\mu\epsilon$, 1057 $\mu\epsilon$, and 1568 $\mu\epsilon$, respectively. The average thermal expansion amplitude on the short edge of the 40 mm deep crack was 1.67 times that of the 20 mm deep crack and 1.48 times that of the 30 mm deep crack. The average thermal expansion amplitudes for the long edge of the cracks with depths of 20 mm, 30 mm, and 40 mm were 873 $\mu\epsilon$, 1035 $\mu\epsilon$, and 1437 $\mu\epsilon$, respectively. The average thermal expansion amplitude on the long edge of the 40 mm deep crack was 1.65 times that of the 20 mm deep crack and 1.39 times that of the 30 mm deep crack. The deeper the crack, the larger the average thermal expansion amplitude for both the short and long edges. This phenomenon is primarily due to the greater cold contraction deformation in rocks with deeper pre-existing cracks, which leads to a larger recovery deformation when the rock temperature returns to room temperature.

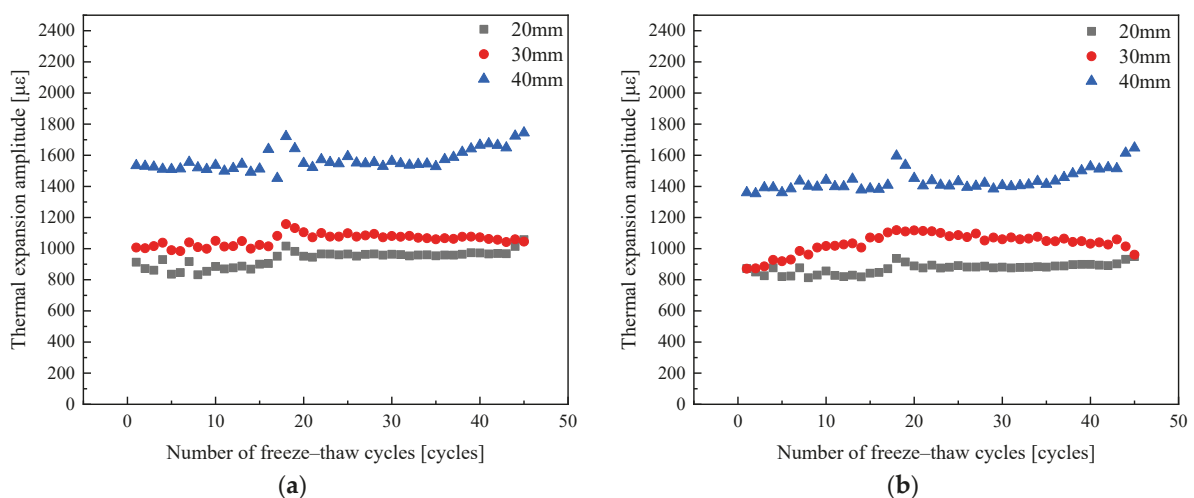


Figure 10. (a) Thermal expansion amplitude of the crack's short edge; (b) thermal expansion amplitude of the crack's long edge.

2. Symmetric Measurement Points on the Short Edge and Long Edge of the Crack

As shown in Figure 11, with the increase in the number of freeze–thaw cycles, the thermal expansion amplitudes of both the short edge and long edge of the cracks in the rock specimens with three different crack depths show minimal changes. The average thermal expansion amplitudes for the short edge of the cracks with depths of 20 mm, 30 mm, and 40 mm are 1022 $\mu\epsilon$, 1379 $\mu\epsilon$, and 900 $\mu\epsilon$, respectively. The 30 mm depth cracked rock specimen exhibits the largest average thermal expansion amplitude for the short edge, followed by the 20 mm depth specimen, with the 40 mm depth specimen showing the smallest amplitude. This indicates that the thermal expansion amplitude variation pattern at the symmetric points of non-penetrating cracks differs from that of the surface points but is generally consistent with the cold contraction amplitude variation pattern. For the long edge, the average thermal expansion amplitudes for the cracks with depths of 20 mm, 30 mm, and 40 mm are 1055 $\mu\epsilon$, 937 $\mu\epsilon$, and 1287 $\mu\epsilon$, respectively. The 40 mm depth cracked rock specimen has the largest average thermal expansion amplitude at the symmetric point on the long edge, followed by the 20 mm depth specimen, with the 30 mm depth specimen showing the smallest amplitude. Regarding the relationship between average thermal expansion amplitude and crack depth, the long-edge symmetric points exhibit the opposite pattern compared to the short-edge symmetric points. This relationship is consistent with that observed for the cold contraction amplitude.

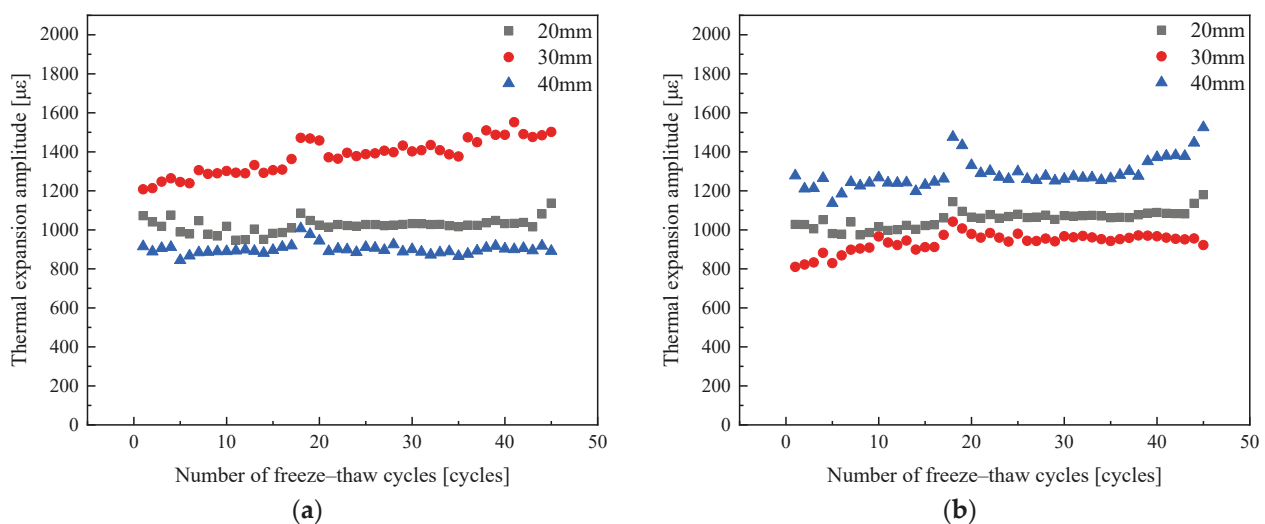


Figure 11. (a) Thermal expansion amplitude at the symmetric point of the crack’s short edge; (b) thermal expansion amplitude at the symmetric point of the crack’s long edge.

3.3.4. Comprehensive Comparative Analysis

- (1) As shown in Figure 12a, in the 20 mm depth cracked rock specimen, both the average cold contraction amplitude and the average thermal expansion amplitude of the short edge are smaller than those of the symmetric points, and the same applies to the long edge.
- (2) As shown in Figure 12b, in the 30 mm depth cracked rock specimen, both the average cold contraction amplitude and the average thermal expansion amplitude of the short edge are smaller than those of the symmetric points, but the average cold contraction and thermal expansion amplitudes of the long edge are greater than those of the symmetric points.
- (3) As shown in Figure 12c, in the 40 mm depth cracked rock specimen, both the average cold contraction amplitude and the average thermal expansion amplitude of the short edge are greater than those of the symmetric points, and the same applies to the long edge.

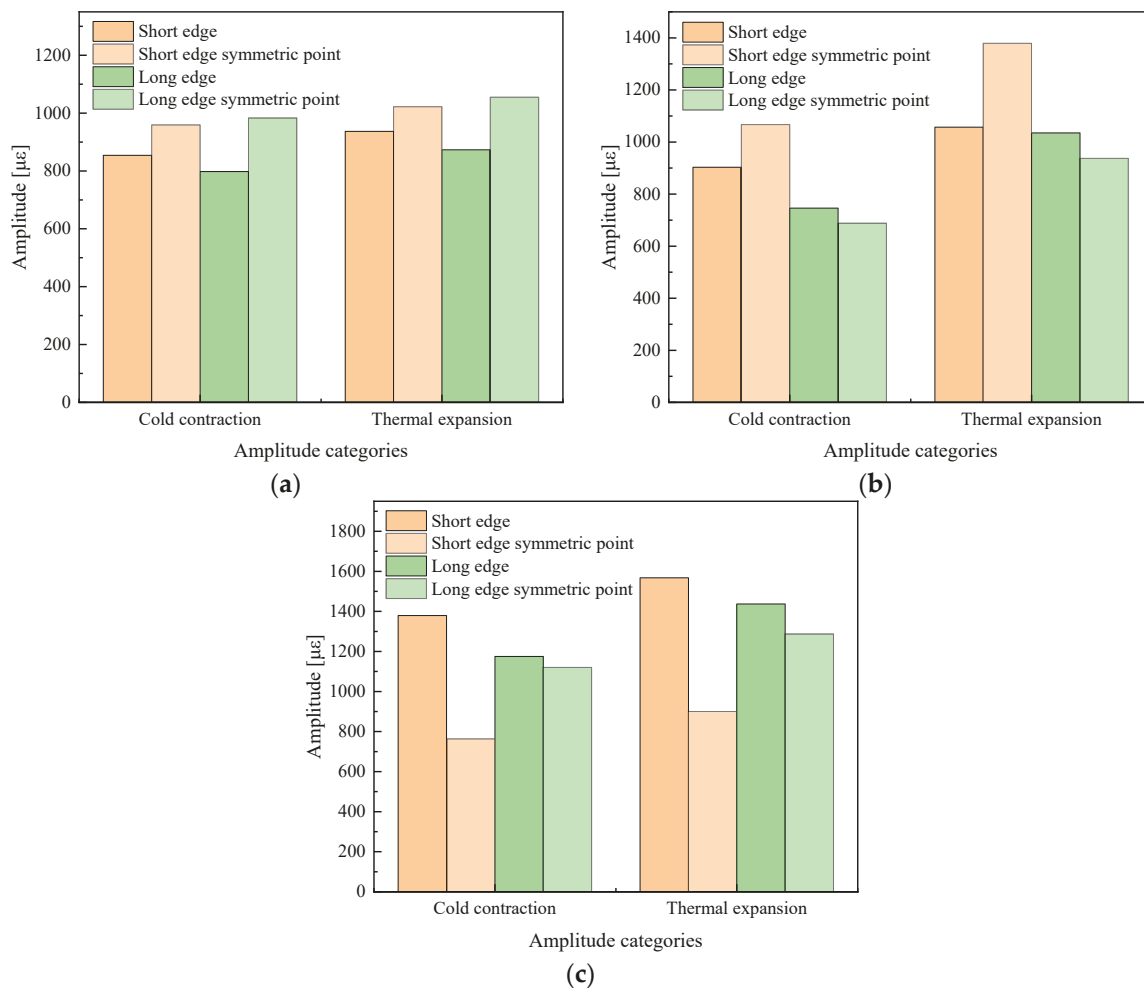


Figure 12. (a) Comparison of strain characteristic values for the 20 mm depth cracked rock specimen; (b) comparison of strain characteristic values for the 30 mm depth cracked rock specimen; (c) comparison of strain characteristic values for the 40 mm depth cracked rock specimen.

In summary, as the depth of the pre-existing crack increases and the depth of the rock matrix at the symmetric points decreases, the average cold contraction and thermal expansion amplitudes of both the short and long edges transition from initially being smaller than those at the symmetric points to ultimately becoming larger than those at the symmetric points. The main reason for this phenomenon may be that as the crack depth increases, the surface points of the pre-existing crack experience less constraint from the surrounding rock matrix. This allows for more pronounced deformation during freeze–thaw cycles. Compared to the constraints at the symmetric points, the degree of freedom at the surface points of the pre-existing crack increases significantly, resulting in the deformation amplitude gradually exceeding that of the symmetric points.

4. Analysis and Discussion

4.1. Analysis of the Micro-Deformation Mechanism of Rock

At different stages of the experiment, a digital microscope was used to observe the microstructures of cracked red sandstone in the dry state, water-saturated state, and after 40 freeze–thaw cycles from two fixed angles, as shown in Figure 13. These microscopic images provide an intuitive representation of the changes in rock skeleton particles during freeze–thaw cycles, highlighting the deformation and damage caused by freeze–thaw action. Additionally, the deformation results revealed by the microscopic images effectively validate the strain data obtained from monitoring experiments.

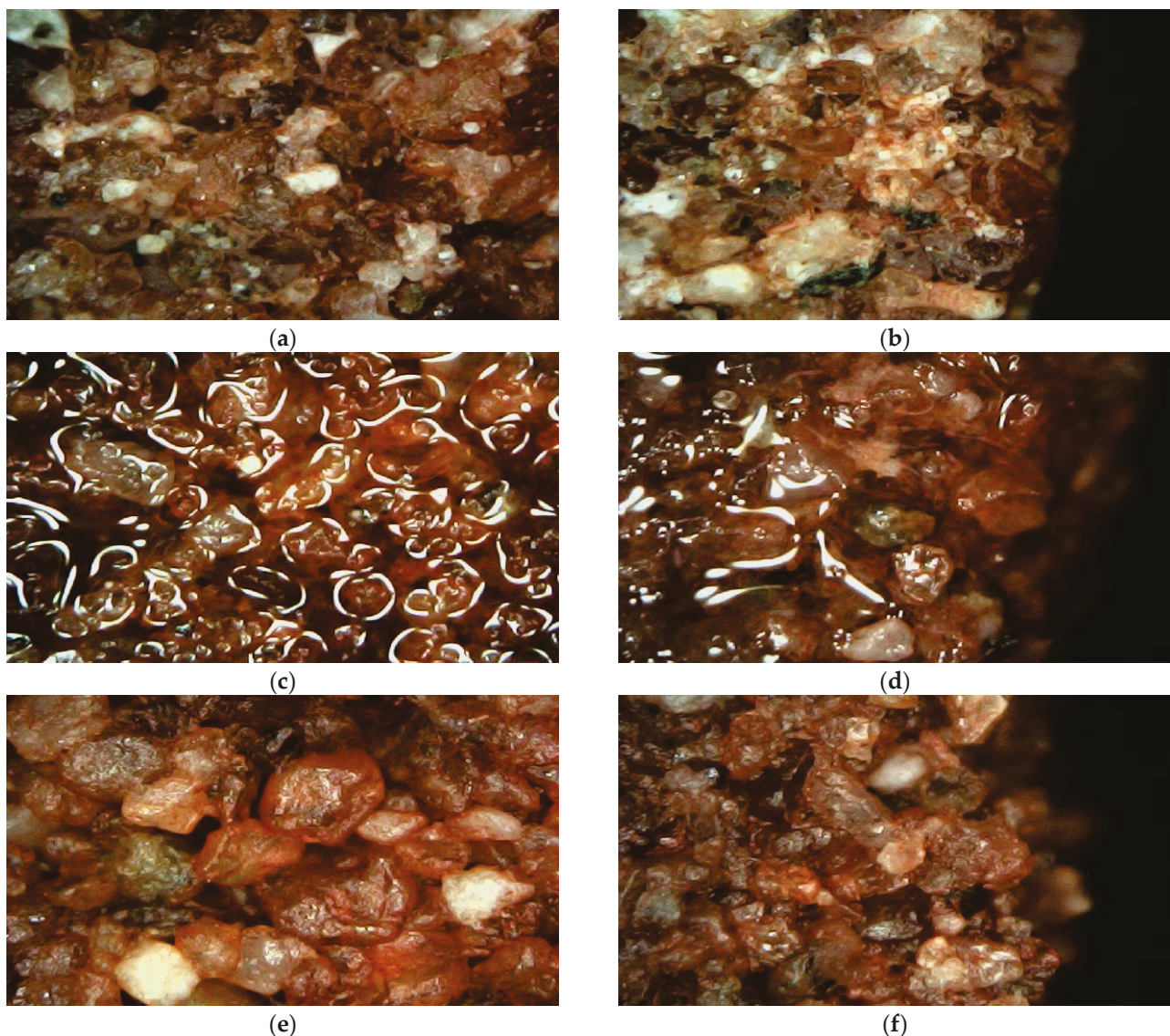


Figure 13. Comparison of (a) 300 $\times$ magnified image of angle 1 in the dry state; (b) 300 $\times$ magnified image of angle 2 in the dry state; (c) 300 $\times$ magnified image of angle 1 in the water-saturated state; (d) 300 $\times$ magnified image of angle 2 in the water-saturated state; (e) 300 $\times$ magnified image of angle 1 after 40 freeze–thaw cycles; and (f) 300 $\times$ magnified image of angle 2 after 40 freeze–thaw cycles.

The red sandstone specimens in the dry state, water-saturated state, and after 40 freeze–thaw cycles exhibit an overall reddish-brown color with uniform particle distribution. In the dry state, the bonding between particles via white debris-like cementing material is clearly visible, with small intergranular pores forming a porous structure. In the water-saturated state, the rock pores are fully filled with water molecules. Comparing microscopic images after 40 freeze–thaw cycles with the initial dried state reveals significant changes. Under the effects of frost heaving forces and thermal expansion and contraction of skeleton particles, the debris-like cementing material sustained considerable damage, leading to a reduction in cementing material. The original pores expanded, the newly formed pores became more numerous, and the remaining skeleton particles were more clearly revealed.

Rock, as a porous medium composed of solid skeleton grains and intergranular pores, has a structure formed by interconnected solid skeleton grains. The pores are usually filled with one or more fluids. The previous literature has clarified that rock deformation is caused by the deformation of skeleton grains and changes in their arrangement [33]. This manifests in two forms, “body deformation” and “structural deformation”, as shown in

Figure 14. The actual deformation of the rock is a combination of these two basic types of deformation, and the total strain is the algebraic sum of the body strain and structural strain. The descriptions of body and structural deformation in rocks are as follows:

- (1) **Body Deformation:** This refers to the overall deformation caused by the deformation of the skeleton grains themselves, which is classified as elastic deformation. During this process, changes in the stress within the skeleton grains lead to changes in their volume, but the arrangement of the skeleton grains remains unchanged. Therefore, the body stress (σ_s) determines the body deformation of the rock.
- (2) **Structural Deformation:** This refers to the overall deformation caused by changes in the arrangement of the skeleton grains, which is classified as plastic deformation. During this process, changes in the contact stress between the skeleton grains result in relative displacement between them, altering the arrangement of the skeleton grains, though the volume of the grains themselves does not change. Therefore, the contact stress (σ_c) between the skeleton grains determines the structural deformation of the rock.

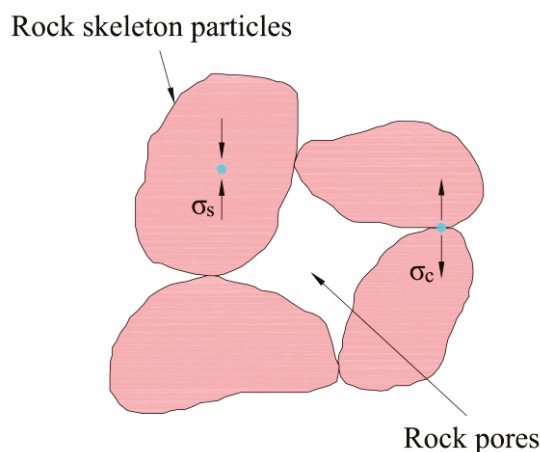


Figure 14. Micro-deformation of rock.

4.2. Discussion

In this experiment, the results of strain variation in cracked rock under freeze–thaw cycles indicate that rock skeleton particles undergo thermal expansion and contraction during the freeze–thaw process. After each freeze–thaw cycle, residual strain and expansion deformation, i.e., plastic deformation, occur. The residual strain accumulates with each cycle, leading to a gradual increase in the overall expansion deformation of the rock as the number of freeze–thaw cycles increases. In summary, the freeze–thaw deformation evolution mechanism of the cracked red sandstone in this experiment can be concluded in the following three points:

- (1) **Thermal Expansion and Contraction of Skeleton Particles:** The rock skeleton particles undergo thermal expansion and contraction during freeze–thaw cycles, resulting in body deformation. Since the skeleton particles are not perfectly elastic, a portion of residual strain remains after each cycle of expansion and contraction. This residual strain constitutes only part of the total residual strain for each freeze–thaw cycle.
- (2) **Frost Heave Force Due to Water Molecules in Rock Pores:** The presence of water molecules in the rock pores leads to approximately 9% volume expansion when the water freezes. If the rock constrains this expansion, a frost heave force is generated [34–36]. This frost heave force affects the contact stress between surrounding skeleton particles, altering their arrangement and causing structural deformation.
- (3) **Damage to Cementing Materials:** In red sandstone, cementing materials bond the surrounding particles together. These materials, being chemical precipitates, are prone to damage from physical and chemical actions. During freeze–thaw cycles, the cementing materials themselves undergo thermal expansion and contraction,

and they are also subjected to external influences such as the compressive effect of expanding skeleton particles, the frost heave force of ice, and the dissolution effects of water molecules. As a result, some of the cementing material gradually breaks down and disappears during the freeze–thaw process. The space it originally occupied transforms into intergranular pores, leading to a significant change in the stress state of the surrounding rock skeleton particles. The contact stress between the particles changes, altering their arrangement and causing structural deformation of the rock.

Additionally, it is worth noting that during the freeze–thaw cycles in this experiment, some of the cementing materials were damaged and disappeared. After the space they originally occupied developed into intergranular pores, the porosity of the rock increased. Since the rock specimens in this experiment were kept in a sealed environment without external water replenishment, the water content of the specimens could not fully fill these newly formed intergranular pores, causing the specimens to become unsaturated. Based on the results of this experiment, the strain variation in the cracked rock and the changes in the strain characteristic values under multiple freeze–thaw cycles did not show significant changes in the later stages of the experiment, and the patterns were largely similar to those observed in the earlier stages. Further research on this issue will be conducted.

5. Conclusions

The present study investigates the deformation behavior of red sandstone with cracks at different depths during freeze–thaw cycles, focusing on the variations in deformation with time and temperature. The freezing–thawing deformation evolution mechanism of cracked red sandstone is analyzed. The main findings are as follows:

- (1) The strain variation process of cracked red sandstone during a single freeze–thaw cycle can be roughly divided into five stages: initial contraction stage, stable contraction stage, continuous contraction stage, thermal expansion stage, and stabilization stage. The strain variation trend can be described as follows: during the cooling and freezing process, the strain in the cracked red sandstone gradually decreases to a minimum value and then stabilizes. During the heating and thawing process, the strain gradually increases to a maximum value, after which it decreases and stabilizes, resulting in residual strain.
- (2) The damage to cracked red sandstone worsens during multiple freeze–thaw cycles. Residual strain, representing plastic deformation, is generated in each freeze–thaw cycle. As the number of freeze–thaw cycles increases, residual strain accumulates, leading to significant expansion deformation and irreversible freeze–thaw damage. The cumulative residual strain of the rock specimen after 45 freeze–thaw cycles was 40.69 times greater than the residual strain from the first cycle. This expansion deformation is severe, manifesting as the extension, increase, and interconnection of rock pores and microcracks.
- (3) The freeze–thaw strain characteristic values of cracked red sandstone include cold contraction amplitude and thermal expansion amplitude, and these values are clearly correlated with crack depth. For the monitoring point on the long edge of the 40 mm depth cracked rock specimen, the average cold contraction amplitude and average thermal expansion amplitude are 1.47 times and 1.65 times those of the 20 mm depth cracked rock specimen, respectively. During freeze–thaw cycles, the rock skeleton particles contract due to the cold. Since the pre-existing crack lacks part of the rock matrix internally, the points at the surface of the pre-existing crack have a slightly higher degree of freedom and are less constrained by the surrounding rock matrix. The deeper the crack, the less constraint at the surface, resulting in more pronounced deformation.
- (4) The freeze–thaw deformation evolution mechanism of cracked red sandstone can be summarized into three points: (i) body deformation caused by the thermal expansion and contraction of rock skeleton particles; (ii) structural deformation caused by frost heave force; and (iii) structural deformation caused by damage to the cementing

materials. This freeze–thaw deformation mechanism provides a valuable reference for the study of the stability and durability of underground rock engineering in cold regions.

This study presents a monitoring method for freeze–thaw deformation, which can be applied to the deformation monitoring of rock underground engineering in cold regions. Additionally, the results of this research provide data references and a mechanistic basis for freeze–thaw damage and deformation in practical underground engineering projects in cold regions.

Author Contributions: Conceptualization, X.H. and G.S.; formal analysis, H.F.; data curation, X.H., C.T. and Z.H.; investigation, P.Y. and Q.Z.; supervision, W.F. and S.Y.; writing—original draft preparation, X.H.; writing—review and editing, G.S.; funding acquisition, G.S. All authors have read and agreed to the published version of the manuscript.

Funding: This work was funded by the National Natural Science Foundation of China (grant number: 51408617), the Scientific Research Project of Hunan Provincial Department of Education (grant number: 22A0189), the General Project of Natural Science Foundation of Hunan Province (grant number: 2024JJ5640), the Hunan Provincial Construction Engineering Science and Technology Research and Development Project (grant number: 23JK162), the Science and Technology Progress and Innovation Plan Project of the Hunan Provincial Department of Transportation (grant number: 202118), and the Science and Technology Project of Henan Provincial Department of Transportation (grant number: 2021-2-12).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: All relevant data are contained in the present manuscript.

Conflicts of Interest: The authors declare no conflicts of interest.

Appendix A

Table A1. List of symbols.

	Symbol
ε_μ	The readings measured by the static resistance strain gauge.
β_c	The coefficient of linear thermal expansion of quartz glass.
Δt	The value of temperature variation.
ε_A	The strain value at point A on the strain–time curve for a single freeze–thaw cycle.
ε_B	The strain value at point B on the strain–time curve for a single freeze–thaw cycle.
ε_C	The strain value at point C on the strain–time curve for a single freeze–thaw cycle.

Table A2. The standard deviation and covariance of the basic physical property parameters of the rock specimens.

Group *	Saturated Density			Water Absorption			Porosity		
	Value [g/cm ³]	Standard Deviation [g/cm ³]	Covariance [%]	Value [%]	Standard Deviation [%]	Covariance [%]	Value [%]	Standard Deviation [%]	Covariance [%]
A	2.156			6.082			13.705		
B	2.264	0.055	2.48	6.349	0.134	2.15	14.295	0.299	2.14
C	2.226			6.235			13.915		

* Taking the cracked rock specimen with a 20 mm crack depth as an example.

Table A3. The standard deviation and covariance of strain during the first freeze–thaw cycle.

Group *	Residual Strain			Cold Contraction Amplitude			Thermal Expansion Amplitude		
	Value [μ ϵ]	Standard Deviation [μ ϵ]	Covariance [%]	Value [μ ϵ]	Standard Deviation [μ ϵ]	Covariance [%]	Value [μ ϵ]	Standard Deviation [μ ϵ]	Covariance [%]
A	13			794			872		
B	12	1.528	11.46	836	40.017	5.03	925	47.127	5.38
C	15			756			831		

* Taking the crack long-edge monitoring point of the cracked rock specimen with a 20 mm crack depth as an example.

Table A4. The standard deviation and covariance of strain during multiple freeze–thaw cycles.

Group *	Residual Strain			Average Cold Contraction Amplitude			Average Thermal Expansion Amplitude		
	Value [μ ϵ]	Standard Deviation [μ ϵ]	Covariance [%]	Value [μ ϵ]	Standard Deviation [μ ϵ]	Covariance [%]	Value [μ ϵ]	Standard Deviation [μ ϵ]	Covariance [%]
A	529			798			873		
B	571	30.79	5.47	830	42.93	5.43	935	48.676	5.52
C	589			745			839		

* Taking the crack long-edge monitoring point of the cracked rock specimen with a 20 mm crack depth as an example.

References

- Xue, Y.; Kong, F.; Yang, W.; Qiu, D.; Ma, X. Main unfavorable geological conditions and engineering geological problems along Sichuan–Tibet railway. *Chin. J. Rock Mech. Eng.* **2020**, *39*, 445–468.
- Chen, H.; Gao, X.; Wang, Q. Research progress and prospect of frozen soil engineering disasters. *Cold Reg. Sci. Technol.* **2023**, *212*, 103901. [CrossRef]
- Yun, M.; Ren, J.; Song, Y.; Zhang, L.; Sun, C.; Chang, P.; Cao, X. Fracture Mechanism and Damage Constitutive Model of Freeze–Thaw Fissured Granite Subjected to Fatigue Loading. *Appl. Sci.* **2024**, *14*, 6324. [CrossRef]
- Ma, W.; Wang, D.Y. Studies on frozen soil mechanics in China in past 50 years and their prospect. *Chin. J. Geotech. Eng.* **2012**, *34*, 625–640.
- Deprez, M.; De Kock, T.; De Schutter, G.; Cnudde, V. A review on freeze–thaw action and weathering of rocks. *Earth Sci. Rev.* **2020**, *203*, 103143. [CrossRef]
- Liu, X.; Congress, S.S.C.; Cai, G.; Liu, L.; Liu, S.; Puppala, A.J.; Zhang, W. Development and validation of a method to predict the soil thermal conductivity using thermal piezocone penetration testing (T-CPTU). *Can. Geotech. J.* **2022**, *59*, 510–525. [CrossRef]
- Maji, V.; Murton, J.B. Micro-computed tomography imaging and probabilistic modelling of rock fracture by freeze–thaw. *Earth Surf. Process. Landf.* **2020**, *45*, 666–680. [CrossRef]
- Yu, W.; Zhang, T.; Lu, Y.; Han, F.; Zhou, Y.; Hu, D. Engineering risk analysis in cold regions: State of the art and perspectives. *Cold Reg. Sci. Technol.* **2020**, *171*, 102963. [CrossRef]
- Yang, C.; Zhou, K.; Xiong, X.; Deng, H.; Pan, Z. Experimental investigation on rock mechanical properties and infrared radiation characteristics with freeze–thaw cycle treatment. *Cold Reg. Sci. Technol.* **2021**, *183*, 103232. [CrossRef]
- Jia, H.; Ding, S.; Zi, F.; Dong, Y.; Shen, Y. Evolution in sandstone pore structures with freeze–thaw cycling and interpretation of damage mechanisms in saturated porous rocks. *Catena* **2020**, *195*, 104915. [CrossRef]
- Liu, T.; Cai, W.; Sheng, Y.; Huang, J. Experimental Study on the Microfabrication and Mechanical Properties of Freeze–Thaw Fractured Sandstone under Cyclic Loading and Unloading Effects. *Materials* **2024**, *17*, 2451. [CrossRef] [PubMed]
- Liu, L.; Cai, G.; Zhang, J.; Liu, X.; Liu, K. Evaluation of engineering properties and environmental effect of recycled waste tire-sand/soil in geotechnical engineering: A compressive review. *Renew. Sustain. Energy Rev.* **2020**, *126*, 109831. [CrossRef]
- Feng, Q.; Yang, Z.; Liu, W.; Zhao, W. Experimental study of the anisotropic frost heave characteristics of rock surrounding tunnels in cold regions. *J. Cold Reg. Eng.* **2021**, *35*, 04021014. [CrossRef]
- Yang, Z.; Xu, J.; Feng, Q.; Liu, W.; He, P.; Fu, S. Elastoplastic analytical solution for the stress and deformation of the surrounding rock in cold region tunnels considering the influence of the temperature field. *Int. J. Geomech.* **2022**, *22*, 04022118. [CrossRef]
- Yu, Q.; Lei, P.; Dai, Z.; Soltanian, M.R.; Yin, S.; Liu, W.; Xiong, Z. Damage characteristics of limestone under freeze–thaw cycle for tunnels in seasonal frozen areas. *Iran. J. Sci. Technol.* **2023**, *47*, 469–477. [CrossRef]
- Tian, Z.; Li, Y.; Wang, G.; Ma, H.; Zhang, J.; Bi, Z. Experimental study on frost heaving force and deformation of water saturated red sandstone fractures. *Chin. J. Rock Mech. Eng.* **2022**, *41*, 2857–2868.
- Jia, P.; Wang, X.; Wang, D. Study on the freeze–thaw deformation characteristics of saturated fractured rocks. *Rock Soil Mech.* **2023**, *44*, 345–354.

18. Kang, Y.; Liu, Q.; Zhao, J.; Zhang, F. Research on frost deformation characteristics of rock and simulation of tunnel frost deformation in cold region. *Chin. J. Rock Mech. Eng.* **2012**, *31*, 2518–2526.
19. Jia, H.; Xiang, W.; Krautblatter, M. Quantifying rock fatigue and decreasing compressive and tensile strength after repeated freeze-thaw cycles. *Permafrost. Periglacial. Process.* **2015**, *26*, 368–377. [CrossRef]
20. Pei, X.; Meng, M.; Yuan, J.; Wang, D. Freezing-thawing characteristics of fractured rockmass under dry and saturated conditions. *Rock Soil Mech.* **2017**, *38*, 1999–2006.
21. Xia, C.; Lv, Z.; Li, Q.; Huang, J.; Bai, X. Transversely isotropic frost heave of saturated rock under unidirectional freezing condition and induced frost heaving force in cold region tunnels. *Cold Reg. Sci. Technol.* **2018**, *152*, 48–58. [CrossRef]
22. Huang, S.; Lu, Z.; Ye, Z.; Xin, Z. An elastoplastic model of frost deformation for the porous rock under freeze-thaw. *Eng. Geol.* **2020**, *278*, 105820. [CrossRef]
23. Lv, Z.; Luo, S.; Xia, C.; Zeng, X. A Thermal-mechanical coupling elastoplastic model of freeze-thaw deformation for porous rocks. *Rock Mech. Rock Eng.* **2022**, *55*, 3195–3212. [CrossRef]
24. Wang, Z.; Zhao, L.; Tu, Z.; Zhang, Z. Thermal expansion measurement at low temperature using strain gauges. *Cryogenics* **1999**, *1*, 18–21.
25. Zhang, H.; Meng, X.; Yang, G. A study on mechanical properties and damage model of rock subjected to freeze-thaw cycles and confining pressure. *Cold Reg. Sci. Technol.* **2020**, *174*, 103056. [CrossRef]
26. Chen, L.; Li, K.; Song, G.; Zhang, D.; Liu, C. Effect of freeze-thaw cycle on physical and mechanical properties and damage characteristics of sandstone. *Sci. Rep.* **2021**, *11*, 12315. [CrossRef]
27. Zhang, X.; Jin, J.; Liu, X.; Wang, Y.; Li, Y. Study on the Influence of Saturation on Freeze-Thaw Damage Characteristics of Sandstone. *Materials* **2023**, *16*, 2309. [CrossRef]
28. Sun, L.; Tao, S.; Liu, Q. Frost crack propagation and interaction in fissured rocks subjected to freeze-thaw cycles: Experimental and numerical studies. *Rock Mech. Rock Eng.* **2023**, *56*, 1077–1097. [CrossRef]
29. Kang, Z.; Wang, Z.; Shao, L.; Feng, J.; Yao, X. Surface crack evolution patterns in freeze-thaw damage of fissured rock bodies. *J. Mt. Sci.* **2024**, *21*, 3094–3107. [CrossRef]
30. Stepień, P.; Spychał, E. The Use of Thermoporometry in the Study of Frost Resistance of Rocks. *Materials* **2024**, *17*, 620. [CrossRef]
31. Dubale, M.; Vasić, M.V.; Goel, G.; Kalamdhad, A.; Singh, L.B. Utilization of construction and demolition mix waste in the fired brick production: The impact on mechanical properties. *Materials* **2022**, *16*, 262. [CrossRef] [PubMed]
32. Dubale, M.; Vasić, M.V.; Goel, G.; Kalamdhad, A.; Laishram, B. The recycling of demolition roof tile waste as a resource in the manufacturing of fired bricks: A scale-up to the industry. *Constr. Build. Mater.* **2024**, *412*, 134727. [CrossRef]
33. Wu, S.; Li, L.; Zhang, X. *Rock Mechanics*, 1st ed.; Higher Education Press: Beijing, China, 2021; Volume 14, p. 430.
34. Lin, H.; Lei, D.; Yong, R.; Jiang, C.; Du, S. Analytical and numerical analysis for frost heaving stress distribution within rock joints under freezing and thawing cycles. *Environ. Earth Sci.* **2020**, *79*, 305. [CrossRef]
35. Qiao, C.; Wang, Y.; Song, Z.; Li, C.; Hou, Z. Experimental study on the evolution characteristics of cyclic frost heaving pressure of saturated fractured granite. *Rock Soil Mech.* **2021**, *42*, 6.
36. Zhang, J.; Wang, G.; Liu, H.; Yang, M.; Ma, C. Characterizing frost heave pressure distribution on rock crack surfaces during freeze-thaw. *Eng. Geol.* **2023**, *324*, 107258. [CrossRef]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Review

Application of Acoustic Emission Technique in Landslide Monitoring and Early Warning: A Review

Jialing Song ^{1,2}, Jiajin Leng ^{1,2}, Jian Li ^{1,2}, Hui Wei ^{2,*}, Shangru Li ³ and Feiyue Wang ⁴

¹ Poly Changda Engineering Co., Ltd., Guangzhou 510620, China; jialingsong@chd.edu.cn (J.S.); lengjiajin1227@126.com (J.L.); jianjiancry234@163.com (J.L.)

² School of Traffic & Transportation Engineering, Changsha University of Science & Technology, Changsha 410114, China

³ School of Civil Engineering, Chongqing Jiaotong University, Chongqing 400074, China; wangan2066@126.com

⁴ School of Civil Engineering, Central South University, Changsha 410083, China; wfyhn@163.com

* Correspondence: wh@csust.edu.cn

Abstract: Landslides present a significant global hazard, resulting in substantial socio-economic losses and casualties each year. Traditional monitoring approaches, such as geodetic, geotechnical, and geophysical methods, have limitations in providing early warning capabilities due to their inability to detect precursory subsurface deformations. In contrast, the acoustic emission (AE) technique emerges as a promising alternative, capable of capturing the elastic wave signals generated by stress-induced deformation and micro-damage within soil and rock masses during the early stages of slope instability. This paper provides a comprehensive review of the fundamental principles, instrumentation, and field applications of the AE method for landslide monitoring and early warning. Comparative analyses demonstrate that AE outperforms conventional techniques, with laboratory studies establishing clear linear relationships between cumulative AE event rates and slope displacement velocities. These relationships have enabled the classification of stability conditions into “essentially stable”, “marginally stable”, “unstable”, and “rapidly deforming” categories with high accuracy. Field implementations using embedded waveguides have successfully monitored active landslides, with AE event rates linearly correlating with real-time displacement measurements. Furthermore, the integration of AE with other techniques, such as synthetic aperture radar (SAR) and pore pressure monitoring, has enhanced the comprehensive characterization of subsurface failure mechanisms. Despite the challenges posed by high attenuation in geological materials, ongoing advancements in sensor technologies, data acquisition systems, and signal processing techniques are addressing these limitations, paving the way for the widespread adoption of AE-based early warning systems. This review highlights the significant potential of the AE technique in revolutionizing landslide monitoring and forecasting capabilities to mitigate the devastating impacts of these natural disasters.

Keywords: landslides; acoustic emission; monitoring systems; early warning; development and challenges

1. Introduction

Landslides are a major global hazard, causing significant socioeconomic losses and over 50,000 fatalities worldwide in the past decade [1]. These destructive events occur in nearly every country, triggered by factors such as heavy rainfall, earthquakes, erosion, deforestation, and human activities like urbanization. Mountainous regions, characterized

by steep terrain and heavy precipitation during the wet season, face a notably higher frequency and risk of landslides [2]. For instance, the southwestern region of China, with its proximity to active seismic zones, experiences a higher prevalence of slope failures (Figure 1). These events endanger human lives and the environment by posing threats to engineering structures, disrupting critical infrastructure, causing structural damage, and contaminating water sources.

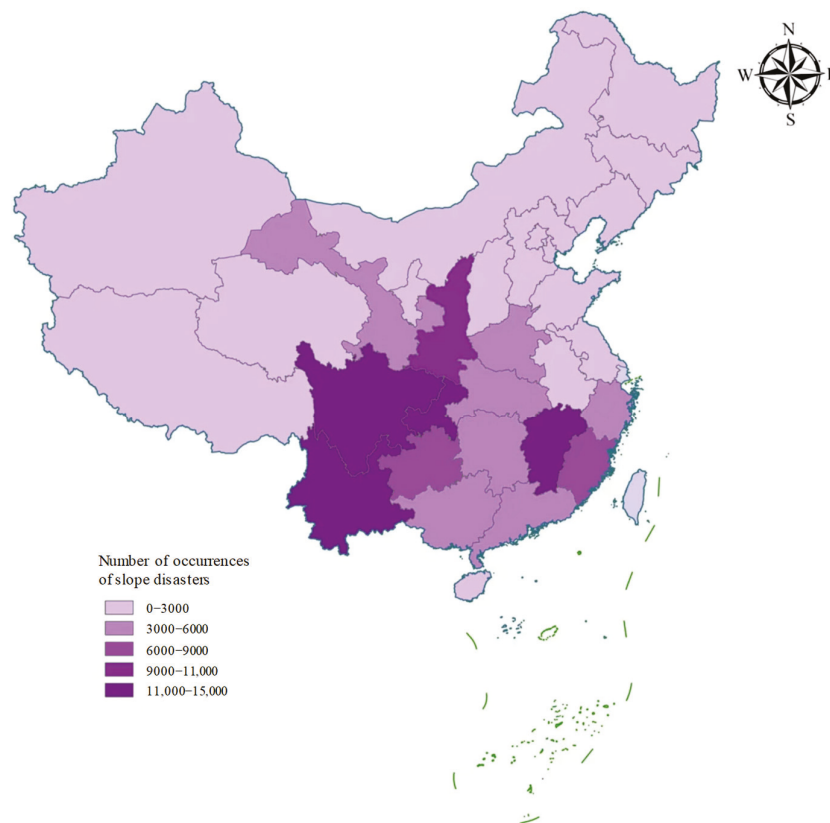


Figure 1. Spatial distribution of slope disasters in China.

Given the complex interplay of factors that contribute to landslides, effective monitoring and early warning systems are essential for mitigating their devastating impacts. The causes of landslides are multifaceted, and involve a combination of short-term triggering factors and underlying predisposing conditions [3,4]. Research has identified a combination of short-term triggering factors and underlying predisposing conditions that contribute to the occurrence of landslides [5]. Major triggering factors include heavy or prolonged rainfall [6], rapid snowmelt [7], changes in groundwater level [8], and earthquakes [9]. Intense rainfall plays a critical role in triggering landslides by weakening the shear strength of soil and rocks through two primary mechanisms. Firstly, it increases the water content in soils and pore water pressure, leading to a decrease in effective stress [10]. This reduction in effective stress makes the soil and rocks more susceptible to failure. Secondly, heavy rainfall adds mass to the already unstable slope material, creating an additional driving force for landslides [11]. The combined effect of increased water content and added mass significantly increases the likelihood of slope failure [12]. In addition to rainfall, rapid snowmelt can also contribute to landslides by saturating the soil and increasing pore water pressure [13]. Changes in groundwater levels can further exacerbate landslide risk by altering the stability of slopes. Earthquakes, with their ground-shaking effects, can directly trigger landslides by destabilizing already vulnerable slopes. To address these challenges,

the implementation of advanced monitoring techniques, such as acoustic emission (AE) monitoring, becomes increasingly necessary.

Landslides often develop gradually, characterized by incremental deformations that culminate in sudden failures [14]. This gradual progression emphasizes the importance of continuous monitoring technologies to detect early signs of destabilization and provide timely warnings [15]. However, the geological heterogeneity of terrain complicates the identification of potential failure zones and the accurate quantification of slope movements. Various surface and subsurface monitoring techniques have been employed to investigate landslide characteristics [16]. Geodetic methods, such as synthetic aperture radar (SAR) [17], aerial/satellite imagery [18], and terrestrial laser scanning (TLS) [19], provide valuable spatial data but are limited in capturing subsurface information. In contrast, geotechnical instrumentation, including inclinometers [20], wire extensometers [21], and time domain reflectometry (TDR) [22], allow for the direct measurement of slope displacements but are restricted to specific monitored areas. Geophysical techniques, including electrical resistivity tomography (ERT) and ground penetrating radar (GPR), offer insights into subsurface structures but require meticulous data processing [23,24]. Conventional measurement methods often detect landslides only after failure has occurred, limiting timely evacuation efforts [25,26]. Therefore, there is a pressing need for monitoring approaches that can continuously capture early warning signals preceding slope collapses.

In this context, acoustic emission (AE) technology emerges as a promising monitoring method. AE captures elastic wave signals generated by stress-induced deformation within materials, offering a novel perspective for landslide monitoring. It can detect micro-damage within rock and soil masses during the early stages of landslide incubation, addressing the limitations of traditional monitoring methods. AE involves the generation of stress wave emissions resulting from irreversible changes in material microstructures under stress, such as friction, cracking, fracturing and melting/crystallization of rocks, resulting in the emission of stress waves [27,28]. Unlike active ultrasonic techniques, AE employs a passive monitoring approach by detecting naturally emitted elastic waves from deforming structures [29]. This enables the faster monitoring of landslides compared to traditional methods. Figure 2 illustrates the schematic diagram of the landslide process and AE monitoring. When integrated with transient signal recording and analysis capabilities, AE monitoring shows potential as an early warning indicator for impending failures in geomaterials [30]. Recent field deployments of AE systems have demonstrated a clear linear relationship between cumulative displacement and AE data [31]. However, challenges remain in capturing signals in highly attenuative soil conditions and establishing precise correlations between AE parameters and deformation states [32].

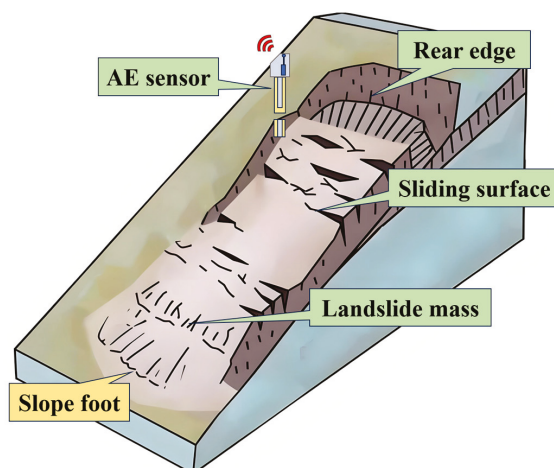


Figure 2. Schematic diagram of landslide process.

This paper aims to offer a thorough review of the latest advancements in AE-based landslide monitoring techniques. After introducing the topic, the paper compares different methods used for monitoring landslides. It then delves into the basic principles and equipment used in the AE technique. The paper also examines how AE testing is applied in studying the stability of granular and rock slopes in both laboratory and field settings. Lastly, it discusses ongoing challenges and recent progress, and provides insights into potential future developments in the field.

2. Current Landslide Monitoring Methods

2.1. Geodetic Methods

Geodetic methods are surface movement monitoring techniques that can be applied over extensive areas, making them particularly useful for landslide investigations. These methods offer the advantage of being able to map large zones and have the potential for satellite-based applications [33]. Three prevalent geodetic techniques include TLS, global positioning systems (GPS), and SAR.

TLS is an active remote sensing method that captures surface topographic data across diverse field conditions [34]. By generating point clouds that represent the geometry of the entire scene, TLS enables the differentiation of terrain changes over time. However, the lack of standardized metrics for precision and accuracy can limit the reliability of the results [35]. Additionally, factors such as incidence angles, surface reflectivity, and ambient light variations can significantly influence scanning quality [36]. Despite these limitations, TLS has proven effective for mapping landslide-induced terrain modifications across various scales, from local to regional levels [37,38].

GPS provides spatial point positioning by triangulating signals from orbiting satellites. This technique measures displacements to assess slope stability by periodically determining the coordinates of reference markers placed within the hazard zone [39,40]. Key strengths of GPS include accuracy levels down to the sub-centimeter range and the capability for autonomous data logging [41]. However, the requirement for an unobstructed view of at least four satellites poses challenges in areas with dense vegetation or deep valleys, which can limit its applicability [42].

SAR functions as an active microwave remote sensing tool that captures terrain echoes in terms of sensor-target distances and azimuth directions during platform flight [43]. This non-contact imaging radar operates effectively under all weather conditions and at any time of day, making it ideal for both short- and long-term slope monitoring without the need for on-site instrumentation [44]. To address atmospheric phase screen effects encountered during monitoring in steep mountainous regions, Karunathilake and Sato [45] proposed a semi-empirical model. However, environmental contaminants, such as atmospheric heterogeneities, can introduce additional noise into the signal [46]. This complicates data processing and interpretation, necessitating advanced techniques to enhance the reliability of SAR data for accurate slope monitoring [47].

In summary, while geodetic methods are effective for mapping landslide shapes and identifying surface movements, they are not suitable for the continuous, real-time monitoring required for effective early warning systems. In complex landslides where failures extend deep underground, these methods provide incomplete information. Furthermore, the high costs and operational requirements associated with establishing geodetic networks in remote or mountainous areas present additional challenges. These methods are also limited in their ability to detect small deformations that may indicate potential slope issues before they escalate, as shown in Table 1.

Table 1. Summary of geodetic methods.

Geodetic Methods	Advantages	Disadvantages
TLS	Active remote sensing; adaptable to diverse field conditions; capable of generating point clouds to identify terrain changes; applicable to mapping landslides at various scales.	Lack of standardized precision metrics; scanning quality affected by incidence angles, reflectivity, and ambient light.
GPS	Provides triangulation positioning via satellite signals; measures displacements to assess stability; offers sub-centimeter-level precision; capable of autonomous data logging.	Requires unobstructed observation of at least four satellites; limited applicability in areas with dense vegetation and deep valleys.
SAR	Active microwave remote sensing; operational in all weather conditions and at all times; does not require on-site installation; suitable for both short-term and long-term monitoring; includes models to mitigate extreme interference.	Affected by environmental pollutants; signal noise complicates data interpretation; data processing is complex.

2.2. Geotechnical Methods

Geotechnical methods utilize in situ instrumentation to conduct targeted monitoring in potential failure zones identified through prior investigations. Key techniques include TDR [48], fiber optic sensing (FOS) [49], inclinometers [50], and settlement gauges [51].

TDR is employed for monitoring landslides by sending electromagnetic pulses through coaxial cables and analyzing reflected signals to detect changes caused by underground movements [52]. This technology tracks landslide activity by measuring moisture fluctuations that indicate unstable conditions. The effectiveness of TDR lies in its ability to transmit and reflect electromagnetic pulses. Over the years, TDR has evolved, with improved installation methods, data interpretation techniques, and simpler data processing approaches [53]. Additionally, it has been integrated with other monitoring technologies to enhance the accuracy and early warning capabilities of landslide monitoring. However, TDR's spatial coverage is limited by the depths and lengths of the installed cables [54].

FOS technology utilizes light signals transmitted through optical fibers to measure varying axial strains resulting from soil deformation [55]. This advanced sensing method allows for continuous strain profiling along the entire length of the sensor, effectively overcoming the coverage limitations of traditional TDR technology. By strategically placing FOS in landslide-prone areas, the real-time monitoring of soil movement and strain can be achieved [56]. These sensors detect physical changes along the optical fiber line through phenomena such as Rayleigh, Brillouin, and Raman scattering, which alter the intensity, phase, and frequency of the scattered light, providing crucial data on temperature and strain [57]. Recent studies have demonstrated that distributed FOS technology is highly effective for landslide monitoring due to its exceptional spatial and temporal resolution, offering nearly instantaneous measurement results. Moreover, FOS is increasingly utilized in long-term monitoring and early warning systems due to its lightweight design, environmental sustainability, resistance to electromagnetic interference, cost-effectiveness, and rapid data transmission capabilities [58]. Nevertheless, operational challenges related to moisture ingress and exposure to sunlight remain significant concerns that need to be addressed [59].

Inclinometers are installed within vertical casing pipes and are used to measure lateral displacements by tracking changes in the inclination of the casing [60]. However, their effectiveness is limited by the difficulty of installation in pre-existing boreholes, restricting their application to known sites. Conversely, settlement gauges are utilized to quantify vertical settlement levels, providing insights into areas experiencing excessive

compression [61]. While both inclinometers and settlement gauges are useful for detecting vertical movements, they are unable to monitor internal shifts occurring beneath the ground surface without causing visible displacements.

The complementarity of these techniques is vital, as no single instrument is adequate for comprehensive monitoring [62]. For example, an integrated network combining GNSS, extensometers, drone photogrammetry, and hydro-meteorological sensors was employed to synergistically investigate the Urbas landslide in northwestern Slovenia [63]. Joint interpretation of FOS and TDR data also enabled differentiation between soil suction and consolidation effects [64].

A significant limitation of geotechnical methods is their inability to capture pre-failure behaviors, as these instruments typically detect displacements only after a triggering event has occurred [65]. Furthermore, the requirement for direct installation in the subsurface may not always be feasible in hazardous zones where failures are incipient or developing. High costs associated with geotechnical networks also render large-scale implementation impractical, particularly when continuous monitoring is essential, such as during heavy rainfall events. Table 2 summarizes the advantages and disadvantages of various geotechnical methods for landslide monitoring.

Table 2. Summary of geotechnical methods.

Geotechnical Methods	Advantages	Disadvantages
TDR	Monitors via electromagnetic pulses; tracks moisture fluctuations; mature technology; integrable with other systems; reliable for long-term monitoring.	Limited spatial coverage based on cable length and depth; installation can be challenging in rugged terrains.
FOS	Measures strain using optical signals; allows continuous strain profile analysis; enables real-time monitoring; offers high spatial and temporal resolution; widely applied due to multiple advantages.	Vulnerable to moisture ingress and damage from UV exposure; initial setup costs can be high.
Inclinometers	Effective for measuring lateral displacements.	Challenging installation; limited application; unable to monitor internal changes underground.
Settlement Gauges	Effectively quantifies vertical settlement; straightforward installation and operation.	Limited to measuring vertical movements; unable to monitor lateral or internal changes.

2.3. Geophysical Methods

Geophysical techniques leverage physical field properties to characterize subsurface conditions in a non-invasive manner, offering significant advantages for landslide investigations [66]. Common applications of these methods include ERT, self-potential (SP), GPR, and downhole seismic techniques.

ERT utilizes variations in electrical resistivity among geological materials to map subsurface structures and properties related to slope instability processes [67]. Resistivity distributions are inferred from surface electrode measurements, facilitated by multichannel systems and inversion modeling. Low resistivity anomalies often indicate developing shear zones due to increased fracturing and pore fluid saturation [68]. However, resistivity values can be influenced by various factors, including temperature, pressure, and the saturation levels of pore fluids. Additionally, the presence of metallic minerals or conductive fluids can adversely affect resistivity measurements.

SP techniques map natural subsurface potential fields generated by fluid and mineral interactions or thermal and electrokinetic coupling associated with slope deformation [69]. This method has been employed to identify preferred directions of landslide movement and areas of seepage. However, temperature variations and electromagnetic field influences can introduce contaminants, complicating the interpretation of SP responses related solely to hydrological factors.

Downhole seismic measurements involve profiling compression and shear wave velocities through boreholes, allowing for a direct assessment of elastic modulus variations within unstable rock and soil masses at different depths [70]. Nonetheless, the effectiveness of this method diminishes with increasing depth due to amplitude attenuation in fractured or weathered zones, where fractures can facilitate weakening. Furthermore, limited access to potential slip zones can restrict the applicability of this methodology.

GPR transmits short-pulse electromagnetic radar signals into the ground to image subsurface features through reflected wave amplitude maps [71]. GPR offers rapid, high-resolution stratigraphic profiling; however, signal penetration can be significantly impaired by water and clay content in the subsurface. The accurate determination of velocity is essential for converting signal data to depth scales, but this is not always feasible under all conditions.

Integrating geophysical techniques with geotechnical and geodetic investigations enhances the characterization of landslides through a unified interpretation approach. AE monitoring can provide insights into the accumulation of preparatory damage before surface displacements become detectable, particularly after surpassing specific thresholds [72]. This method effectively captures internal micro-event emissions, reducing signal attenuation. By combining datasets from various scales and domains, a comprehensive understanding of landslide evolution can be achieved, ultimately leading to improved early warning systems. Table 3 summarizes the strengths and limitations of geophysical methods.

Table 3. Summary of geophysical methods.

Geophysical Methods	Advantages	Disadvantages
ERT	Maps subsurface using resistivity differences; infers distribution with multichannel systems and modeling; identifies shear zone anomalies.	Influenced by temperature, pressure, and saturation levels; measurements can be interfered with by metallic minerals and conductive fluids.
SP	Maps natural subsurface potential fields; identifies directions of landslide movement and seepage areas.	Temperature and electromagnetic fields can introduce interference; limits clear interpretation of hydrological factors.
Downhole Seismic Techniques	Measures wave velocity in boreholes to determine elastic modulus; directly assesses subsurface stability.	Effectiveness decreases with increasing depth; limited access to potential slip zones can restrict application.
GPR	Transmits signals for imaging, providing rapid and high-resolution profile measurements; effective for identifying subsurface features.	Signal penetration is affected by water and clay content; velocity determination may not always be achievable.

2.4. Remote Sensing Methods

Remote sensing utilizes aerial or satellite-based photogrammetric imaging platforms to delineate areas at risk of landslides [73]. Key techniques include SAR interferometry and the interpretation of optical satellite imagery.

SAR is an active microwave remote sensing technology mounted on satellite platforms and capable of capturing images day and night, regardless of weather conditions [74]. It generates surface backscatter by transmitting microwave signals and measuring the distances of echoes and antenna flight bearing angles. Differential SAR interferometry (DInSAR) analyzes phase differences between interferometric SAR image pairs to map ground deformation with millimeter accuracy by comparing reference and current acquisitions [75]. However, DInSAR requires the precise co-registration of images and correction of atmospheric artifacts, which can be challenging for operational applications. Additionally, frequent imaging repeats are necessary to dynamically monitor continuously evolving slopes.

Passive optical sensors acquiring multispectral data enable morphological terrain characterization through supervised classification routines assigning each pixel into distinct zones linked to terrain/slope features [76]. High-resolution commercial satellites such as IKONOS, QuickBird, and WorldView constellation facilitate detailed feature identification in stereoscopic compositions [77]. Change detection for identifying areas of significant slope gradient/stability variations through pre-/post-event comparisons is another popular application.

Despite its successful applications, remote sensing faces several challenges in its role within landslide early warning systems [78]. Real-time monitoring is hindered by the intermittent revisit intervals of satellites and the availability of data acquisition, which are influenced by unpredictable factors such as weather conditions, thereby limiting the time available for preparedness [79]. Although multi-temporal data can aid in identifying past landslide occurrences, directly measuring preparatory slope deformations is often impractical. Furthermore, surface features may not accurately reflect the subsurface failure mechanisms controlled by hidden geological characteristics.

In general, remote sensing enhances traditional field mapping by providing a comprehensive overview that is conducive to advanced territorial zoning, retrospective analysis of detailed landslide histories, and identification of changes following major events. However, the limitations in dynamically monitoring evolving slope movements in real-time necessitate the integration of subsurface characterization methods. This integration can supplement remote sensing data and observations with on-site geophysical datasets capable of detecting precursory deformations. Table 4 provides advantages and limitations of remote sensing methods.

Table 4. Summary of remote sensing methods.

Remote Sensing Methods	Advantages	Disadvantages
SAR and DInSAR	Active microwave remote sensing, capable of imaging around the clock under all weather conditions; DInSAR can measure deformation with millimeter-level accuracy and conduct dynamic monitoring of slopes.	Difficulties in image registration and atmospheric correction can complicate analysis; frequent imaging is required, leading to high operational costs and significant demands on data processing.
Passive Optical Sensors	Characterizes terrain morphology using multispectral data; high-resolution satellite imagery allows for detailed feature identification through stereoscopic compositions; detects changes to identify areas with slope variations.	Difficult to achieve real-time monitoring; data acquisition is affected by satellites and weather conditions; unable to directly measure pre-deformation of slopes; surface features hardly reflect the subsurface instability mechanisms.

2.5. Limitations of Existing Methods

Although current methods for monitoring landslides have demonstrated success in identifying hazardous areas and analyzing past incidents, they possess limitations that hinder their effectiveness as reliable early warning systems. Table 5 summarizes the advantages and disadvantages of several traditional slope monitoring methods.

Table 5. Comparison of slope monitoring methods.

Monitoring Methods	Advantages	Disadvantages
Geodetic Methods	High precision; direct measurement; less affected by weather.	Limited measurement points; low efficiency; high requirements for the environment.
Geotechnical Methods	Strong pertinence; real-time monitoring; combined with engineering practice.	Damage to the rock and soil mass; high maintenance costs; representativeness issues.
Geophysical Methods	Large-area detection; non-contact measurement; multi-parameter measurement.	Damage to the rock and soil mass; high maintenance cost; representativeness issues
Remote Sensing Methods	Macroscopic monitoring; fast data update; not restricted by terrain.	Relatively low precision; dependent on weather and lighting; complex data processing.
AE Technique	Early damage sensitivity; possession of a natural signal source; applicability in multiple fields; comprehensive analysis of signal parameters.	Potential damage to rock and soil masses; high maintenance costs; issues with representativeness.

Geodetic techniques are constrained by the intermittent nature of data acquisition, which is not suited for the continuous monitoring required for unstable slopes. Additionally, surface measurement techniques fail to capture subsurface kinematics that are critical for understanding buried structures, infrastructure, and processes such as deep-seated soil creep. Furthermore, the interpretation of the results often involves significant uncertainties due to factors such as vegetation density, atmospheric distortions, and terrain irregularities.

Geotechnical instrumentation faces challenges related to limited subsurface coverage, as the costly deployment of extensive networks along every potential failure plane at sufficient depths is often impractical. Post-failure detection can delay timely responses, even if full slope coverage is achieved. The high costs associated with these systems prohibit large-scale implementations without continuous government subsidies, which is unsustainable for developing countries, which experience the majority of global landslide disasters each year. Additionally, field instrumentation can become ineffective when access to hazardous areas is obstructed during active failure events.

Geophysical methods are limited by resolution constraints and may yield ambiguous results, necessitating integration with additional techniques, as standalone solutions often lack clearly interpretable parameters. Seismic monitoring requires dense station networks for effective source localization, which increases costs and is not feasible over extensive areas.

Remote sensing methods are impaired for continuous monitoring due to infrequent revisit periods and dependence on favorable weather conditions for image acquisition. Detected changes typically correspond to displacements that have already occurred, rather than capturing early-stage precursory deformations. Moreover, discrepancies may exist between the surface morphological expressions delineated from optical imagery and the actual shear planes that govern internal slope failure mechanics.

These limitations render existing methods less effective as early warning tools that can be integrated into community-based disaster preparedness strategies. Such strategies ideally require low-cost sensors that are robust enough for on-site autonomous operations, coupled with reliable analytical models to provide reasonable lead times for response. Additionally, attenuation remains a major challenge, impeding the direct sensing of granular slope deformation by electromagnetic, acoustic, or mechanically coupled waves. An ideal approach must identify strategies to overcome these restrictions, and the AE technique holds promising capabilities in this regard, as will be highlighted subsequently.

3. Fundamentals of Acoustic Emission Technique

3.1. Acoustic Emission Phenomenon

AE denotes the emission of elastic stress waves resulting from sudden stress release due to irreversible internal structural changes within materials [80]. These stress waves propagate through the material as two main types: compressional (primary) waves, characterized by alternating compressions and rarefactions moving in the same direction as the wave propagation, and shear (secondary) waves, which entail particle movement perpendicular to the energy transport direction [81].

AE testing relies on the passive detection of spontaneously emitted transient elastic waves from the surface or subsurface of a structure (body) using transducers, which convert these waves into electrical signals through the piezoelectric effect. These signals are then amplified for recording and analysis [82]. Unlike ultrasonic testing, which transmits mechanical waves artificially, AE sources arise naturally from micro-failures induced by service or testing within the monitored system (structure, component, or ground mass). This constitutes an active non-destructive testing methodology that is sensitive to early damage development, as shown in Figure 3.

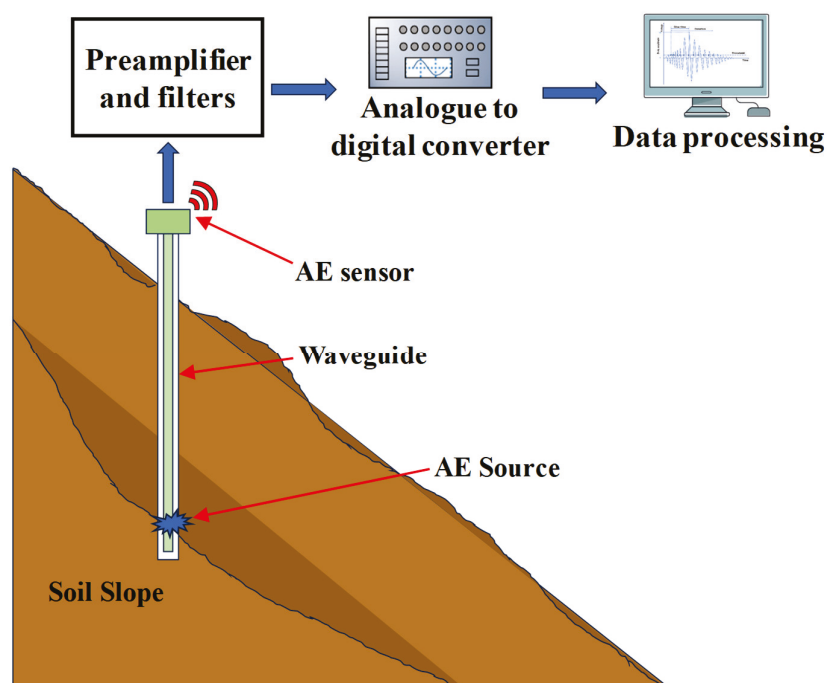


Figure 3. Working principle diagram of AE sensor.

Numerous micro- and macro-level processes involving localized stress redistributions are considered potential mechanisms for AE generation in soils and rocks [83]. Inter-particle friction, sliding, rolling, plowing, and collisions occur during shearing or complex loading of granular materials [84]. Inherent heterogeneities facilitate the formation of

force chain networks, where load transfers concentrate on narrow contacts experiencing slip-stick fluctuations under changing boundary conditions. These fluctuations are closely associated with localized elastic wave releases. In both laboratory and field measurements, brittle cracking, crack–crack interactions, crack growth acceleration, and coalescence within monolithic rock materials are widely documented as sources of microseismic AE [85]. Furthermore, fracture processes at material interfaces such as joints, fault zones, cementation planes, and clay-rich laminae also emit AE signals [86].

In landslide monitoring, body waves and surface waves both produce AE, depending on their orientation relative to sensor placement [87]. However, body waves typically predominate as the primary component of the failure process due to their higher velocities. Attenuation refers to the gradual reduction in amplitude and energy as waves propagate outward from the AE source. This attenuation is primarily caused by geometric damping resulting from wavefront expansion, material intrinsic absorption that converts mechanical energy into heat, and scattering at internal interfaces that alters wave particle movements. The significant variation in attenuation observed in geological materials has important implications for the detection range of AE, highlighting the need to consider propagation paths when designing AE monitoring systems.

3.2. Instrumentation for Acoustic Emission Monitoring

Critical AE instrumentation components encompass sensors as transducers, pre-amplifiers, main amplifiers, filters, and an analog-to-digital converter linked to a data acquisition system for parameter extraction and analysis [88]. Figure 4 summarizes the schematic illustration of AE instrumentation.

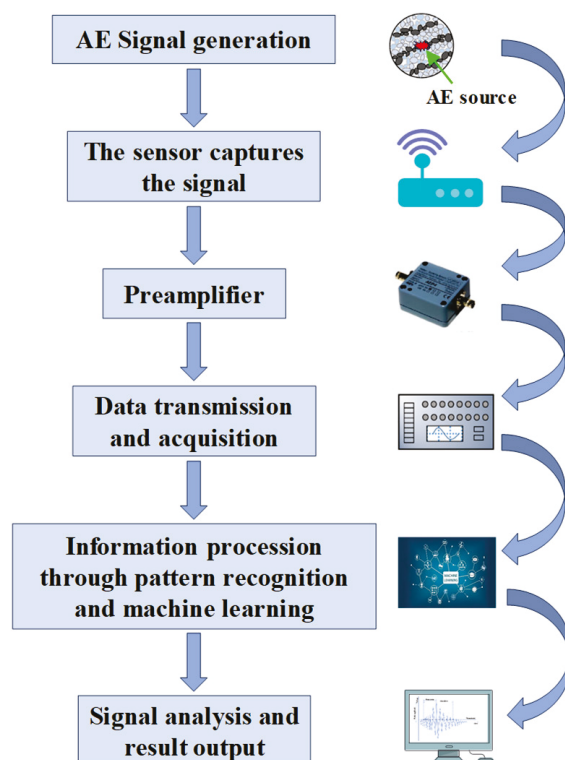


Figure 4. Schematic illustration of AE instrumentation.

Highly sensitive broadband sensors are favored for capturing a broad range of frequencies. Piezoelectric wafers made from lead zirconate titanate (PZT) and polarized during manufacturing are widely utilized because they convert mechanical strains induced by elastic waves into voltage signals through crystal deformations [89]. Sensors vary in

their frequency ranges, spanning from 0.1 kHz to over 1 MHz, with resonance frequencies adjusted according to specific applications and the properties of the monitored materials.

Sensors must establish effective coupling with monitored structures, either by direct mounting on surfaces or through waveguides, especially in media with high attenuation such as soils [90]. The skin effect, arising from the wavelength of AE exceeding crystal thickness, leads to surface responses that benefit from flush mounting on surfaces to ensure contact with a coupled fluid layer that resonates with the structure for optimal transmission. Coupling materials should possess acoustic impedance similar to that of both sensors and structures to minimize reflections that could weaken signal penetration into the test structures and enhance sensitivity [91].

Signals typically remain weak at nano- and microvolt levels, necessitating pre-amplification to enhance amplitudes before transmission through coaxial cables to main amplifiers positioned at a distance from the sensors to mitigate electromagnetic interferences [92]. Pre-amplifiers with gains ranging between 20 and 40 dB amplify signals to microvolt and millivolt levels, making them suitable for further amplification by main multi-channel amplifiers located far from the sensors. In comparison to traditional cable sensors, optical fiber AE sensors offer several advantages, including high sensitivity, broad frequency response, immunity to electromagnetic interference, capability to operate in extreme temperatures, remote monitoring, and intrinsic safety [93].

Filtering cleans signals from electromagnetic noise, environmental disturbances, and high/low frequency artifacts, respectively, before digitization [94]. High pass filters above 20–30 kHz eliminate seismic/acoustic noises, while low pass filters under 1 MHz attenuate resonance peaks centered at sensor natural frequencies to retrieve genuine signals. Adjustable gain amplifiers regulate final signal amplitudes before analog-to-digital conversion.

Multi-channel data acquisition systems connect amplifiers to computers running customized software for additional processing, event discrimination, and parameter extraction [95]. Fast Fourier transformations are used to analyze frequency domain spectra from time-domain signals, while continuous wavelet transforms extract time–frequency information by converting signals into coded digital bitstream formats [96]. Data analysis provides parameters such as amplitudes, energy levels, rise times, counts, and event rates for empirical correlations with mechanical and failure responses observed through reference techniques [97].

Overall, well-designed AE systems enhance sensitivity by combining components that work together effectively as integrated units to account for the unique attenuation characteristics of geological materials. This ensures signal integrity throughout the entire process, from detection to analysis. Furthermore, the incorporation of automated algorithms for pattern recognition and machine learning modeling improves the interpretation of data from growing datasets collected during continuous monitoring periods.

3.3. Parameters of Acoustic Emission Signals

AE signals consist of transient elastic waves that propagate through materials as a localized response to rapid stress and strain energy releases associated with irreversible damage events occurring internally. As non-stationary stochastic signals, AE signals are primarily analyzed through the extraction of parameters in the time, frequency, and time–frequency domains for empirical quantification [98].

In the time domain, essential features of AE are characterized by the detected waveforms and envelopes, as illustrated in Figure 5 and summarized in Table 6.

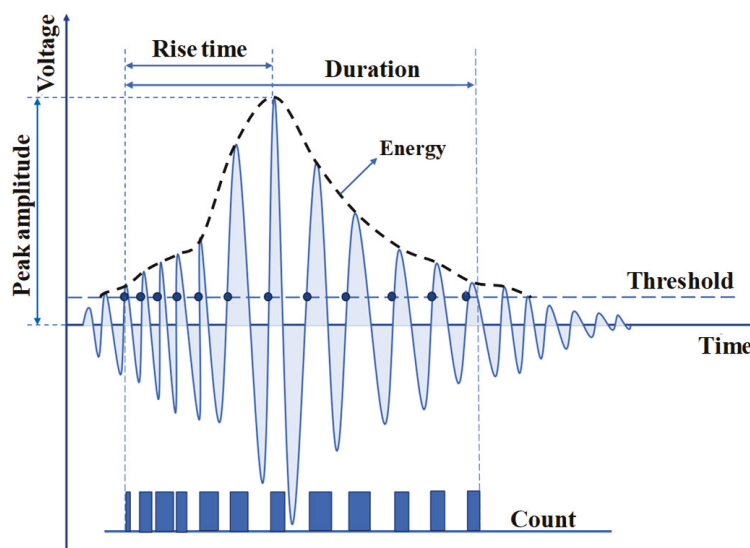


Figure 5. Parameters defined for a typical AE waveform.

Table 6. Parameters of acoustic emission signals.

Parameter Name	Definition	Function or Significance
Amplitude	The intensity of the peak signal, measured in decibels relative to a reference value or in volts, depending on sensor characteristics.	Represents the energy released by the seismic source and the degree of attenuation during the propagation process.
Rise Time	The time interval from when the signal first exceeds the initial threshold to when it reaches peak amplitude.	Reflects the rate of deformation energy release; a short rise time indicates a sudden release of energy (e.g., brittle intergranular fracture), while a longer rise time suggests a more gradual process, such as ductile yielding.
Count	The number of times the signal amplitude exceeds a preset threshold level within a user-defined time window.	Affected by the amplitude and frequency of the wave; serves as a proxy for the amount of energy released.
Duration	The time interval from the first instance the signal exceeds the threshold to the last instance it does.	Represents the total duration of the signal packet when a discrete event is detected; a longer duration indicates contributions from multiple sources during the event.
Energy	A parameter obtained by integrating the square of the instantaneous voltage signal over the entire duration of the event.	Quantitatively represents the total magnitude of energy released during the event.

Moreover, advanced time–frequency domain analyses using wavelet transforms allow for the simultaneous extraction of signal components across a broad frequency range, capturing changes in dominant frequency contributions over time [99]. This technique effectively tracks the dynamic evolution of the dominant frequency in the time dimension, presenting a comprehensive and accurate portrayal of the signal’s frequency characteristics at different moments. This provides crucial data support for studying the developmental patterns and internal mechanisms of the signal. L.M. Spasova et al. have validated the effectiveness and reliability of this method in their research on acoustic emission signals generated during the melting and crystallization of granites [100]. Compared to Fourier analysis alone, this approach is particularly adept at revealing temporal variations in signals

with non-stationary characteristics. Parameters such as wavelet energy, cross-correlation coefficients, and standard deviation are valuable for quantitatively comparing signals and serve as references for qualitative interpretations [101].

Empirical observations have demonstrated distinct correlations and sensitivities among various AE parameters in relation to different failure mechanisms in soils and rocks [102]. For example, AE energy, duration, and frequency centroid exhibit greater sensitivity to shear fractures in rock joints compared to amplitude and rise time. Du et al. [103] concluded that energy and peak frequency are effective indicators of rock crack growth processes. Different loading modes also result in varying predominant parameters. Therefore, careful recognition of the significance of these parameters is essential for tailored quantitative analyses of specific source processes in geomaterials.

3.4. Frequency Characteristics of Acoustic Emission in Soils

The frequency characteristics provide critical insights into underlying AE generation mechanisms in soils as they are directly related to stress/strain waveforms released over source process durations. However, observed AE frequency ranges vary widely depending on the soil types, test conditions, and analysis methods employed, as shown in Figure 6.

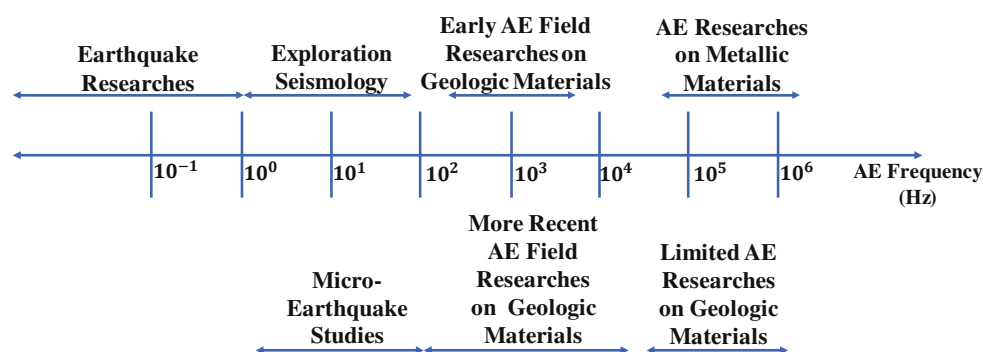


Figure 6. Frequency range for acoustic emission monitoring.

Several studies have shown that AE signals from granular soils span a 0.1–1000 kHz range in general [29]. Particle scale interactions including sliding, rolling, and breakage produce higher frequency emissions. For example, single-particle crushing tests using silica/coral sands identified dominant frequencies within 100–600 kHz corresponding to particle failure timescales [104]. Similarly, tropical residual soil fracture occurred at 100 Hz–20 kHz under triaxial conditions [105].

Soil sample shearing experiments indicate that sliding and rolling contacts emit lower frequencies. Results from Dagois-Bohy et al. [106] monitoring avalanching sand flows found dominant 60–160 Hz emissions attributed to grain collisions and friction. Similarly, direct shear tests on glass beads sheared at 60 kPa produced up to 80 kHz AE signals [107]. Lower confined stresses facilitated more grain reorganization, releasing lower energy signals at lower frequencies.

Unsaturated soil testing is less reported. But results indicate an influence of the degree of saturation on detectable frequency ranges. Frequencies up to 10 kHz were recorded during partially saturated sand flow experiments, presumably owing to localized pore structure reconfigurations accompanying fluid redistributions [108].

Regarding field applications, typically lower monitoring frequencies prove practical. Landslide analyses employed seismometers capturing 20–30 Hz MS signals from slipping soil/rock volumes [109,110]. Likewise, excavation slope assessments relied on 5–8 kHz bandwidth detections through embedded waveguides [111]. Lower frequency ranges allow

for the monitoring of larger volumes hampered at higher bands by high attenuation in geological media.

Overall, multi-frequency analysis may better characterize failure micro-mechanisms in granular soils by capturing the co-existence of grain-scale and fluid–pore interactions, depending on frequency sensitivities. Additionally, factors like stress-states, saturation conditions, and soil mineralogy introduce complexity, necessitating systematic parametric investigations for establishing quantitative correlations between frequency contents and specific processes driving slope instability, relevant for monitoring and early warning objectives.

3.5. Attenuation Characteristics of Acoustic Emission in Soils

Attenuation refers to the exponential decay in AE signal amplitude as it propagates away from its source location within a material. It poses a major impediment for employing AE techniques for long-range subsurface monitoring applications in geo-materials exhibiting high damping capacities. Understanding attenuation properties specific to soils proves crucial to optimize instrumentation for overcoming restricted detection limits.

Attenuation arises due to factors like geometric spreading, scattering, mode conversion, material intrinsic damping from internal friction, and macro/micro structural heterogeneities [112]. It depends jointly on frequency, confining stresses, and soil mineralogical-textural attributes through their influence on elastic moduli and fabric anisotropy.

Ono [113] investigated the frequency-dependent performance of ultrasonic transducers and AE sensors, demonstrating that the reception sensitivity can be precisely evaluated by utilizing both sinusoidal wave and impulse excitation techniques. At lower test frequencies, the impact of attenuation significantly rises as a result of the escalating sensor impedance. Consequently, adjustments are necessary to determine the intrinsic sensitivity of a sensor, also known as open-circuit sensitivity, which may deviate by over 20 dB from outcomes obtained with typical preamplifiers featuring an input impedance of around 10 k Ω .

Koerner et al. [114] reviewed data on AE attenuation in different geomaterials like sands, silts, gravels, and intact rocks. Sands demonstrated the highest damping, in the order of 10 dB/cm, which decreased geometrically with increasing effective confining stress from 10 dB/cm at 70 kPa to 2 dB/cm at 700 kPa. Attenuation for silts and gravels ranged within 1–10 dB/cm. Figure 7 shows the relationship between AE attenuation and frequency in sandy soil.

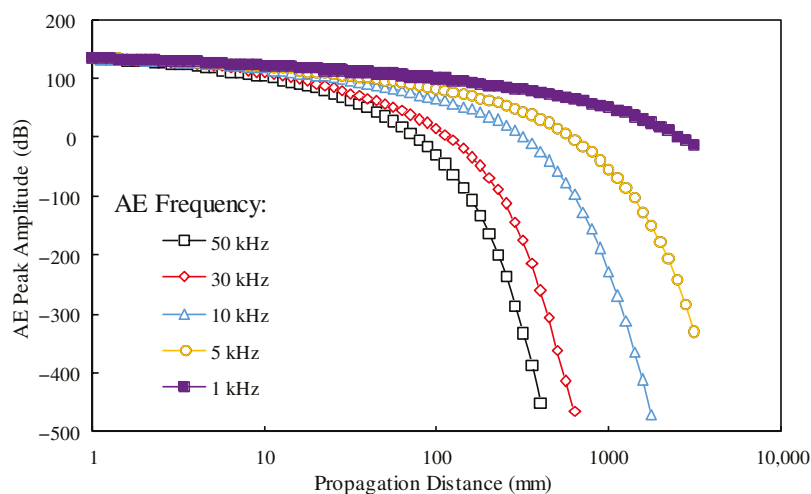


Figure 7. AE attenuation and frequency in sandy soil.

Lin et al. [115] discovered that in saturated soils, the interconnected isotropic water phase promotes the transmission of acoustic waves with reduced attenuation, resulting in an increased detection of AE events. In contrast, in dry or unsaturated soils, the propagation of acoustic waves becomes more intricate due to the existence of discontinuities like particle–particle contacts and interfaces between particles and air or water. Consequently, fewer AE signals are identified compared to saturated soils, and the detection sensitivity is influenced by the frequency spectrum of the emitted waves, as varying frequencies experience diverse levels of attenuation during transmission.

Establishing correlations between AE attenuation and geotechnical parameters, García-Ros et al. [116] indicated decreasing trends with increasing consolidation stress and void ratio. While attenuation increased hyperbolically with frequency, moisture content exponentially amplified attenuation.

Attenuation mechanisms remain complex combinations of scattering from void space heterogeneities, grain-to-grain contact impedance mismatches, intrinsic material losses, and moisture influences coupling dissipative fluid motions [117]. Accordingly, reliable attenuation predictions demand calibrated models accounting for predominant physics through multivariable dependencies.

Overall, significant soil attenuation poses a significant challenge for long-range AE monitoring endeavors. Future studies that combine empirical validations with microstructure-informed numerical simulations incorporating accurate grain characteristics, spatial arrangements, and soil moisture conditions offer potential for predictive attenuation assessments. This approach can enhance the optimization of field sensor placement and the design of instruments tailored to particular geological materials and stress conditions.

4. Discussion

4.1. Field Monitoring of Soil Slopes

The primary goal of implementing AE monitoring in soil slopes is supported by foundational laboratory validations [109]. Early implementations encountered significant obstacles due to substantial wave attenuation in porous, unconsolidated soil masses, making remote detections impractical. To address these challenges, effective strategies utilize waveguides to enable localized self-emission generation and transmission, particularly along predefined embedded routes.

Initial attempts monitored three active landslides in Colorado, USA, using steel waveguides in trench beds recording AE counts through portable recorders over months [118]. Despite technical limitations, relationships between increased AE and seasonal slope movements were qualitatively interpreted. Using similar setups, Hardy Jr and Taioli [119] reported thresholds for evaluating stability through establishing activity gradients classified as low, moderate, and high.

Subsequent studies optimized waveguide configurations. Steel tubes backfilled with sands and gravels as “active” waveguides transmitted emissions from deforming host soils towards attached sensors along borehole sections [120]. Cowden and Arseley coastal slopes were monitored quantitatively, establishing empirical proportionalities linking event rates linearly to displacement velocities between 0.001 and 1 mm/min for failure forecasting.

The choice of waveguides for AE monitoring is crucial, as it involves balancing coupling efficiency with flexibility. The selection process is also influenced by the monitoring system’s overall effectiveness. Aluminum waveguides outperformed steel ones in transmission efficiency, primarily due to their better compatibility with soil conditions owing to lower seismic impedance mismatch [110].

Recent advancements have consolidated quantification schemes. Linear relationships describe normalized AE event rates directly scaling with real-time displacement rates reproducibly for triaxial modeling, landslide flume, and field slope experiments [121]. Soil-type-dependent proportionality coefficients were proposed to enable early warnings based on gradient criteria of “essentially stable”, “marginally stable”, “unstable”, and “rapidly deforming”, according to linearly extrapolated velocities from AE trends.

Multi-location arrays also located AE epicenters, validating that sources corresponded to slope shear planes and not noise. Slope deformation fronts and zones were successfully mapped using portable arrays of passive detectors linked to centralized recorders for spatially distributed sensing along potential failure courses ahead of morphological imprints left along displaced paths [122]. Data analytics established kinematic fingerprints like b-value variations indicative of preparatory damage build-ups facilitating slope behavior forecasting.

Overall, AE provided subsurface non-contact access addressing major constraints of traditional geotechnical inclinometer readings strictly limited to post-failure stages. Well-designed active waveguides capable of generating localized measurable self-emissions from incipient deforming soils underpinned a fully fledged early warning system validated through decade-long developmental research and numerous successful landslide premonitory detections.

4.2. Monitoring of Rock Slopes

Rock slopes experience failures differently from soil slopes due to anisotropic lithological variations determining strength and disturbance zones depending on discontinuity orientations. Brittle cracks nucleate under stress perturbations, permitting sudden collapses once thresholds are surpassed. Successful AE applications provide insights into failure precursors, potentially forecasting impending breakdowns.

Investigations were performed to examine cliff instabilities [123]. Five seismic stations monitored chalk cliffs in Normandy, France, observing rising activity hours preceding collapse interpreted as progressive damaging. Three months of monitoring of unstable columns in Vercors massif limestone cliffs distinguished fracture process regimes from microseismicity [124].

Large-scale mining-induced slope failures motivated the monitoring of collapsed opencast coal mines in Indonesia using seismic sensors [125]. Back-analyses established empirical relationships between precursor bursts and unstable mass dimensions, permitting predictive modeling simulating failure propagation sequences and outcomes.

Laboratory studies established correlations between AE parameters emitted during true-triaxial tests and failure processes within Aztec sandstone [126]. Results suggest the feasibility of employing AE techniques for seismic hazard evaluation from earthquakes, mining-induced seismicity, dynamite explosions, etc.

Waveguides offered the means of embedding multiple sensors inside boreholes for enhanced detection. Wireless earthquake detection devices incorporated fiber-optic waveguides transmitting Raman backscatter-modulated strain signals from fractured rock masses [127]. Simulations showed the potential for structural health monitoring as damage signals were resolved.

Recent studies have utilized a convolutional neural network (CNN) to automatically classify seismic signals collected over a 15-year period on the Åknes rock slope in Western Norway [128]. The CNN effectively distinguished and categorized eight types of events, achieving an accuracy rate of approximately 80%. This analysis yielded important findings regarding the microseismic activity at Åknes, which exhibited pronounced seasonality correlated with annual temperature fluctuations. By automating the classification process,

this research makes a notable advancement in landslide monitoring, enabling the real-time analysis of seismic events associated with slope movements.

Overall, rock slope monitoring through borehole-embedded passive/active AE waveguides furnished direct access into internal damage evolution when sub-surface interrogation was not possible through surface techniques. Emerging machine learning capabilities promise augmented quantitative understandings linking the onset and growth of precursor emissions to slope kinematics, improving forecast lead-times in urgent demand for advanced warning applications.

4.3. Quantification of Acoustic Emission Signals

Quantifying AE parameters into physical unit representations provides a crucial rationalization permitting the interpretation of monitored responses as real deformation indicators. Early investigations qualitatively classified activity levels as shown below:

1. Low: below 1000 AE counts with a low probability of failure [129].
2. Moderate: 1000–10,000 counts indicating marginal stability requiring monitoring [118].
3. High: 10,000–100,000 counts, representing an unstable state demanding remediation [119].
4. Very high: above 100,000 counts, signaling failure onset necessitating evacuation [118].

Nevertheless, establishing AE parameters as objective indicators correlated with externally measured deformation rates has been essential for the development of failure prediction models that incorporate specific warning thresholds, with the following capabilities.

1. Establishing linear scaling between the natural logarithm of the AE event rate (normalized by duration) and monitored landslide flume displacement rates [121].
2. Rating slope stability into four categories of “essentially stable”, “marginally stable”, “unstable”, and “rapidly deforming” through gradient classification of scaled AE–displacement rate relation [121].
3. Demonstrating proportional correlation between normalized cumulative AE energy and measured daily displacements of slow-sliding landslide monitored using optical leveling for validation [109].

In addition to quantification, damage classification and the identification of deforming zones have utilized pattern recognition tools, in the following ways:

1. Through identifying slope instabilities from stable regions through cluster analyses of AE dataset attributes like b-value, partition coefficient, and skewness [130].
2. Through the spatio-temporal tracking of slope structural failure processes through Hidden Markov Models, decomposing AE sequences into discrete states signifying damage evolution stages [122].

Overall, quantitative indices constitute the main focus of contemporary AE monitoring towards enabling dependable early warnings through establishing objective success/failure criteria scientifically validated against reference measurements. In the long run, advanced analytical techniques like machine learning may revolutionize data-driven quantitative evaluations of monitored slope health.

5. Challenges and Recent Advances

5.1. Signal Detection Under High Attenuation Conditions

Attenuation-induced amplitude decay impedes distant subsurface signal propagation, presenting a formidable challenge for long-range in situ monitoring applications employing the AE technique. Overcoming high damping capacities exhibited by geological media constitutes an active area of ongoing research and development.

Key strategies focus on optimizing hardware designs and embedded sensor configurations tailored for specific material conditions to enhance sensitivity and detection efficacy under severe signal weakening scenarios. Notable developments include the following:

1. Employing wideband piezoelectric transducers preferentially tuned to lower frequency ranges resulted in reduced damping compared to higher bands [113].
2. Deploying localized denser multi-sensor arrays configured in advanced geometries like concentric circles and nested quadrants to maximize detection through coincidence logic combining signals [131].
3. Coupling sensors and waveguides firmly using optimized viscoelastic materials with acoustic impedances closely matching both for minimal reflection losses and maximized transmission [132].
4. Implementing advanced low-noise pre-amplifier designs integrated very close to sensors to boost initial gains, countering losses through early-stage amplification [133].

Overall, hardware customized according to specific propagation environments and application depths coupled with expertise in strategic sensor configurations holds immense potential for realizing long-range subsurface AE monitoring aspirations hitherto restricted by attenuation limitations.

5.2. Improvements in Sensing and Data Acquisition

Advancements in sensor technologies and data recording systems increasingly aid in higher-efficacy subsurface AE monitoring under adverse conditions. Notable progresses encompass the following:

1. Micro-electro-mechanical system-based ultra-compact sensors with broad 20 kHz to 1 MHz bandwidths, low self-noise, resonance frequencies up to 1 GHz, and diameters less than 1 mm for dense embedding [112].
2. Fiber-optic-distributed acoustic sensing utilizing Rayleigh backscatter in optical fibers for continuous stroke monitoring over extended lengths, circumventing signal attenuation limitations of traditional point sensors [134].
3. Wireless battery-powered low-power sensor nodes streamlining sensor arrays through autonomy, avoiding cabling and overcoming field installation complexities [135].
4. Cloud-hosted data repositories augmenting computation and storage capacities facilitating advanced pattern recognition, machine learning, and visualization tools, empowering data-driven interpretation [136].

In general, continuous progress in sensing and data acquisition systems holds the potential to enhance the capabilities of AE techniques for the more accurate monitoring of subsurface defects across large areas, even in challenging conditions. This evolution includes the development of advanced quantification and autonomous methodologies.

5.3. Developments in Signal Analysis Techniques

Contemporary analytics frameworks increasingly exploit expanding datasets from optimized acquisition platforms. Prominent developments include the following:

1. Advanced statistical tools analyzing spatio-temporal attributes like b-value distributions, inter-event times, covariance, and clustering, identifying damage progression signatures;
2. Machine learning algorithms automating pattern recognition through supervised classifiers discerning stable/unstable slope conditions and source characteristics;
3. Deep neural networks trained on massive labeled datasets, achieving source localization and forecasting slope behavior through predictive modeling, often outcompeting conventional linear regression techniques;

4. Numerical wavefield simulations incorporating virtual complex structures and sensor layouts, optimizing hardware designs through design validation prior to deployments under the guidance of full-waveform analyses.

Current advancements are moving towards the integration of augmented datasets with physics-based comprehensive numerical–statistical frameworks, leveraging increasing computational capabilities for precise quantitative data-driven damage assessment and prognostics. When coupled with optimized data acquisition systems, these developments hold great promise for enhancing long-range subsurface monitoring with high accuracy.

5.4. Integrating Acoustic Emission with Other Technologies

Leveraging complementary attributes of multiple techniques presents a pragmatic approach overcoming limitations of individual methods. Hybrid monitoring schemes integrating AE hold immense potential for comprehensive evaluations.

Combining AE with traditional geodetic methods addresses uncertainties from isolating surface displacements. Cross-correlating underground fracture growth from AE activity with ground-based interferometric SAR deformation patterns augmented slope stability assessments through validated subsurface–surface couplings [15].

The fusion of AE parameters with continuous hydrological monitoring through in situ pore-pressure sensors quantitatively enabled the discrimination of dilatant failure regions from stable compressive zones, as directly evidenced through hydraulic head changes [137]. Establishing damage–hydraulic couplings optimized predictive modeling by enabling physics-based extrapolations beyond calibration sites.

Exploiting mutual complementary attributes between techniques like wider spatial/temporal coverage of geodetic/hydrological methods against the subsurface sensitivity of AE presents an effective paradigm for advanced long-term predictive monitoring schemes deployed in operational warning systems. Hybrid schemes promise the overcoming of limitations inherent to the isolation of single modalities optimizing monitoring scopes. Combined with optimized signal processing and machine learning frameworks, future applications utilizing such integrated multi-technology platforms appear most promising.

6. Conclusions and Future Perspectives

This study reviewed the fundamentals and state-of-the-art applications of the AE technique for landslide monitoring and early warning. Key conclusions and future outlooks are as follows.

1. The AE method exploits the high-resolution, non-contact sensing of elastic emissions generated by evolving subsurface damage as soils and rocks deform. While laboratory investigations have established qualitative and quantitative understandings of the failure mechanisms, signal attenuation remains the foremost challenge for long-range field implementations. To address this, ongoing innovations are optimizing embedded sensor configurations, waveguides, and hardware designs tailored to the specific propagation conditions.
2. Advancements in MEMS/optical sensing, data acquisition systems, and wireless automated networks are transforming monitoring capabilities towards higher fidelities over extensive volumes with continuous coverage. However, validating the evolving quantification schemes under diverse geological settings remains necessary to establish their reliability and widespread applicability.
3. The integration of AE monitoring with multimodal sensing techniques, such as surface deformation measurements and hydrological monitoring, has enhanced the comprehensive characterization of subsurface failure mechanisms. By leveraging the

complementary attributes of individual methods, these hybrid monitoring frameworks overcome the limitations of isolating single modalities, thereby optimizing the monitoring scope and improving the accuracy of predictive models.

In light of these findings, this review recommends future research to explore machine learning-based analysis techniques for AE signals, which may significantly enhance predictive capabilities and improve data interpretation. Considering the complexity of AE data and their correlations with various landslide-related factors, data-driven approaches can help uncover intricate signal features and patterns that are difficult to detect through traditional analysis methods, enabling the more accurate prediction of landslide occurrences. Additionally, the development of coordinated multi-scale monitoring frameworks, optimized through virtual simulations, can further advance the integration of AE with other sensing technologies for enhanced landslide forecasting capabilities.

Overall, the AE technique holds great promise for achieving the long-standing goal of comprehensively characterizing subsurface slope responses through quantitative early precursory detections. This capability can substantially augment disaster mitigation efforts by providing reliable early warning systems. Continued research and development, particularly in the context of climate change and the increasing frequency of extreme weather events, is essential for strengthening the effectiveness of AE as a robust solution for landslide prediction.

Author Contributions: Conceptualization, J.S. and J.L. (Jiajin Leng); methodology, J.L. (Jian Li); software, S.L.; validation, J.L. (Jian Li), H.W. and F.W.; formal analysis, J.S.; investigation, J.S. and J.L. (Jiajin Leng); resources, F.W.; data curation, J.L. (Jiajin Leng) and S.L.; writing—original draft preparation, J.S. and S.L.; writing—review and editing, H.W. and F.W.; visualization, J.L. (Jian Li); supervision, H.W.; project administration, J.S.; funding acquisition, H.W. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the National Natural Science Foundation of China, grant numbers 52278436 and 52208426; the National Key R&D Program of China, grant number 2023YFB2603500; the Scientific Research Project of Hunan Provincial Department of Education for Key Research, grant number 24A0241; the Science and Technology Innovation Program of Hunan Province, grant number 2022RC1024; and the Foundation of Engineering Research Center of Catastrophic Prophylaxis and Treatment of Road and Traffic Safety of Ministry of Education, grant number kfj220402.

Data Availability Statement: Not applicable.

Conflicts of Interest: Authors Jialing Song, Jiajin Leng and Jian Li were employed by the company Poly Changda Engineering Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

References

1. Froude, M.J.; Petley, D.N. Global fatal landslide occurrence from 2004 to 2016. *Nat. Hazards Earth Syst. Sci.* **2018**, *18*, 2161–2181. [CrossRef]
2. Zhang, Y.; Ayyub, B.M.; Gong, W.; Tang, H. Risk assessment of roadway networks exposed to landslides in mountainous regions—A case study in Fengjie County, China. *Landslides* **2023**, *20*, 1419–1431. [CrossRef]
3. Maki Mateso, J.-C.; Biellers, C.L.; Monsieurs, E.; Depicker, A.; Smets, B.; Tambala, T.; Bagalwa Mateso, L.; Dewitte, O. Characteristics and causes of natural and human-induced landslides in a tropical mountainous region: The rift flank west of Lake Kivu (Democratic Republic of the Congo). *Nat. Hazards Earth Syst. Sci.* **2023**, *23*, 643–666. [CrossRef]
4. Zhou, Z.; Shen, J.-H.; Li, Y.; Duan, W.-F.; Yang, R.-C.; Shu, J.-C.; Li, H.-W.; Tao, S.-Y.; Zheng, S.-Z. Mechanism of colluvial landslide induction by rainfall and slope construction: A case study. *J. Mt. Sci.* **2021**, *18*, 1013–1033. [CrossRef]
5. Ding, C.; Feng, G.; Liao, M.; Tao, P.; Zhang, L.; Xu, Q. Displacement history and potential triggering factors of Baige landslides, China revealed by optical imagery time series. *Remote Sens. Environ.* **2021**, *254*, 112253. [CrossRef]

6. Komolvilas, V.; Tanapalungkorn, W.; Latcharote, P.; Likitlersuang, S. Failure analysis on a heavy rainfall-induced landslide in Huay Khab Mountain in Northern Thailand. *J. Mt. Sci.* **2021**, *18*, 2580–2596. [CrossRef]
7. Xian, Y.; Wei, X.; Zhou, H.; Chen, N.; Liu, Y.; Liu, F.; Sun, H. Snowmelt-triggered reactivation of a loess landslide in Yili, Xinjiang, China: Mode and mechanism. *Landslides* **2022**, *19*, 1843–1860. [CrossRef]
8. Liu, Q.; Jian, W.; Nie, W. Rainstorm-induced landslides early warning system in mountainous cities based on groundwater level change fast prediction. *Sustain. Cities Soc.* **2021**, *69*, 102817. [CrossRef]
9. Qiu, H.; Su, L.; Tang, B.; Yang, D.; Ullah, M.; Zhu, Y.; Kamp, U. The effect of location and geometric properties of landslides caused by rainstorms and earthquakes. *Earth Surf. Process. Landf.* **2024**, *49*, 2067–2079. [CrossRef]
10. Liang, W.-L. Dynamics of pore water pressure at the soil–bedrock interface recorded during a rainfall-induced shallow landslide in a steep natural forested headwater catchment, Taiwan. *J. Hydrol.* **2020**, *587*, 125003. [CrossRef]
11. Chang, C.; Wang, G. Creep of clayey soil induced by elevated pore-water pressure: Implication for forecasting the time of failure of rainfall-triggered landslides. *Eng. Geol.* **2022**, *296*, 106461. [CrossRef]
12. Ivanov, V.; Arosio, D.; Tresoldi, G.; Hojat, A.; Zanzi, L.; Papini, M.; Longoni, L. Investigation on the role of water for the stability of shallow landslides—Insights from experimental tests. *Water* **2020**, *12*, 1203. [CrossRef]
13. Friele, P.; Millard, T.H.; Mitchell, A.; Allstadt, K.E.; Menounos, B.; Geertsema, M.; Clague, J.J. Observations on the May 2019 Joffre Peak landslides, British Columbia. *Landslides* **2020**, *17*, 913–930. [CrossRef] [PubMed]
14. Zhang, J.; Tang, H.; Li, C.; Gong, W.; Zhou, B.; Zhang, Y. Deformation stage division and early warning of landslides based on the statistical characteristics of landslide kinematic features. *Landslides* **2024**, *21*, 717–735. [CrossRef]
15. Casagli, N.; Intrieri, E.; Tofani, V.; Gigli, G.; Raspini, F. Landslide detection, monitoring and prediction with remote-sensing techniques. *Nat. Rev. Earth Environ.* **2023**, *4*, 51–64. [CrossRef]
16. Le Breton, M.; Bontemps, N.; Guillemot, A.; Baillet, L.; Larose, É. Landslide monitoring using seismic ambient noise correlation: Challenges and applications. *Earth-Sci. Rev.* **2021**, *216*, 103518. [CrossRef]
17. Mondini, A.C.; Guzzetti, F.; Chang, K.-T.; Monserrat, O.; Martha, T.R.; Manconi, A. Landslide failures detection and mapping using Synthetic Aperture Radar: Past, present and future. *Earth-Sci. Rev.* **2021**, *216*, 103574. [CrossRef]
18. Li, H.; He, Y.; Xu, Q.; Deng, J.; Li, W.; Wei, Y. Detection and segmentation of loess landslides via satellite images: A two-phase framework. *Landslides* **2022**, *19*, 673–686. [CrossRef]
19. Jiang, N.; Li, H.; Hu, Y.; Zhang, J.; Dai, W.; Li, C.; Zhou, J.-W. A monitoring method integrating terrestrial laser scanning and unmanned aerial vehicles for different landslide deformation patterns. *IEEE J. Sel. Top. Appl. Earth Obs. Remote Sens.* **2021**, *14*, 10242–10255. [CrossRef]
20. Ruzza, G.; Guerriero, L.; Revellino, P.; Guadagno, F.M. A multi-module fixed inclinometer for continuous monitoring of landslides: Design, development, and laboratory testing. *Sensors* **2020**, *20*, 3318. [CrossRef]
21. Ciampalini, A.; Farina, P.; Lombardi, L.; Nocentini, M.; Taurino, V.; Guidi, R.; Pina, F.d.; Tavarini, D. Integration of satellite inSAR with a wireless network of geotechnical sensors for slope monitoring in urban areas: The Pariana landslide case (Massa, Italy). *Remote Sens.* **2021**, *13*, 2534. [CrossRef]
22. Devendra, K.Y.; Guntha, K.; Singam, J.; Santos, K.D.; Sanjay, K.S. Studying Time Domain Reflectometry to Predict Slope Failure in Open-Cast Mines. *J. Min. Sci.* **2020**, *56*, 760–770. [CrossRef]
23. Ullah, F.; Lijun, S.; Cheng, L.; Zhenyu, L.; Bingli, H. Geophysical prospecting related to earthflow reactivation and hazard assessment: A study based on Huangnibazi slope failure in Sichuan Province, China. *Bull. Eng. Geol. Environ.* **2022**, *81*, 21. [CrossRef]
24. Wróbel, M.; Stan-Kłeczek, I.; Marciniak, A.; Majdański, M.; Kowalczyk, S.; Nawrot, A.; Cader, J. Integrated geophysical imaging and remote sensing for enhancing geological interpretation of landslides with uncertainty estimation—A case study from Cisiec, Poland. *Remote Sens.* **2022**, *15*, 238. [CrossRef]
25. Whiteley, J.; Watlet, A.; Uhlemann, S.; Wilkinson, P.; Boyd, J.; Jordan, C.; Kendall, J.; Chambers, J. Rapid characterisation of landslide heterogeneity using unsupervised classification of electrical resistivity and seismic refraction surveys. *Eng. Geol.* **2021**, *290*, 106189. [CrossRef]
26. Zakaria, M.T.; Mohd Muztaza, N.; Zabidi, H.; Salleh, A.N.; Mahmud, N.; Rosli, F.N. Integrated analysis of geophysical approaches for slope failure characterisation. *Environ. Earth Sci.* **2022**, *81*, 299. [CrossRef]
27. Wei, H.; Liu, Y.; Li, J.; Wang, F.; Zheng, J.; Yuan, Z. Characterizing fatigue damage evolution in asphalt mixtures using acoustic emission and Gaussian mixture model analysis. *Constr. Build. Mater.* **2023**, *409*, 133973. [CrossRef]
28. Ojovan, M.I.; Gibb, F.G. Exploring the Earth’s crust and mantle using self-descending, radiation-heated, probes and acoustic emission monitoring. In *Nuclear Waste Research: Siting, Technology and Treatment*; Nova Science Publishers, Inc.: Hauppauge, NY, USA, 2008; pp. 207–220.
29. Mao, W.; Hei, L.; Yang, Y. Advances on the acoustic emission testing for monitoring of granular soils. *Measurement* **2021**, *185*, 110110. [CrossRef]

30. Liu, Z.-m.; Jiang, Y.; Wang, D.-j.; Fu, Y.-j. Four types of acoustic emission characteristics during granular stick-slip evolution. *J. Mt. Sci.* **2022**, *19*, 276–288. [CrossRef]
31. Deng, L.; Yuan, H.; Chen, J.; Zhang, M.; Su, G.; Zhou, Y.; Chen, Y. Experimental investigation and field application of acoustic emission array for landslide monitoring. *Landslides* **2024**, *21*, 71–81. [CrossRef]
32. Kumar, D.; Mahapatro, A.K.; Singh, S.K. Active waveguide deformation dynamics using acoustic emission technology for landslide early warning system. *Bull. Eng. Geol. Environ.* **2024**, *83*, 68. [CrossRef]
33. Sestras, P.; Bilaşco, Ş.; Roşca, S.; Veres, I.; Ilies, N.; Hysa, A.; Spalević, V.; Cîmpeanu, S.M. Multi-instrumental approach to slope failure monitoring in a landslide susceptible newly built-up area: Topo-Geodetic survey, UAV 3D modelling and ground-penetrating radar. *Remote Sens.* **2022**, *14*, 5822. [CrossRef]
34. Kermarrec, G.; Yang, Z.; Czerwonka-Schröder, D. Classification of terrestrial laser scanner point clouds: A comparison of methods for landslide monitoring from mathematical surface approximation. *Remote Sens.* **2022**, *14*, 5099. [CrossRef]
35. Jiang, N.; Li, H.-B.; Li, C.-J.; Xiao, H.-X.; Zhou, J.-W. A fusion method using terrestrial laser scanning and unmanned aerial vehicle photogrammetry for landslide deformation monitoring under complex terrain conditions. *IEEE Trans. Geosci. Remote Sens.* **2022**, *60*, 4707214. [CrossRef]
36. Birien, T.; Gauthier, F. Assessing the relationship between weather conditions and rockfall using terrestrial laser scanning to improve risk management. *Nat. Hazards Earth Syst. Sci.* **2023**, *23*, 343–360. [CrossRef]
37. Lissak, C.; Bartsch, A.; De Michele, M.; Gomez, C.; Maquaire, O.; Raucoules, D.; Roulland, T. Remote sensing for assessing landslides and associated hazards. *Surv. Geophys.* **2020**, *41*, 1391–1435. [CrossRef]
38. Weidner, L.; Walton, G. Monitoring the effects of slope hazard mitigation and weather on rockfall along a Colorado highway using terrestrial laser scanning. *Remote Sens.* **2021**, *13*, 4584. [CrossRef]
39. Ma, Y.; Li, F.; Wang, Z.; Zou, X.; An, J.; Li, B. Landslide assessment and monitoring along the Jinsha river, Southwest China, by combining InSAR and GPS techniques. *J. Sens.* **2022**, *2022*, 9572937. [CrossRef]
40. Luo, W.; Dou, J.; Fu, Y.; Wang, X.; He, Y.; Ma, H.; Wang, R.; Xing, K. A novel hybrid LMD–ETS–TCN approach for predicting landslide displacement based on GPS time series analysis. *Remote Sens.* **2022**, *15*, 229. [CrossRef]
41. Wang, P.; Liu, H.; Nie, G.; Yang, Z.; Wu, J.; Qian, C.; Shu, B. Performance evaluation of a real-time high-precision landslide displacement detection algorithm based on GNSS virtual reference station technology. *Measurement* **2022**, *199*, 111457. [CrossRef]
42. Fiolleau, S.; Uhlemann, S.; Falco, N.; Dafflon, B. Assessing probability of failure of urban landslides through rapid characterization of soil properties and vegetation distribution. *Geomorphology* **2023**, *423*, 108560. [CrossRef]
43. Nava, L.; Monserrat, O.; Catani, F. Improving landslide detection on SAR data through deep learning. *IEEE Geosci. Remote Sens. Lett.* **2021**, *19*, 4020405. [CrossRef]
44. Tsokas, A.; Rysz, M.; Pardalos, P.M.; Dipple, K. SAR data applications in earth observation: An overview. *Expert Syst. Appl.* **2022**, *205*, 117342. [CrossRef]
45. Karunathilake, A.; Sato, M. Atmospheric phase compensation in extreme weather conditions for ground-based SAR. *IEEE J. Sel. Top. Appl. Earth Obs. Remote Sens.* **2020**, *13*, 3806–3815. [CrossRef]
46. Wang, Y.; Dong, J.; Zhang, L.; Zhang, L.; Deng, S.; Zhang, G.; Liao, M.; Gong, J. Refined InSAR tropospheric delay correction for wide-area landslide identification and monitoring. *Remote Sens. Environ.* **2022**, *275*, 113013. [CrossRef]
47. Belhadj-aissa, S.; Simard, M.; Jones, C.E.; Oliver-Cabrera, T.; Christensen, A. Separation of water level change from atmospheric artifacts through application of independent component analysis to InSAR time series. *Earth Space Sci.* **2024**, *11*, e2024EA003540. [CrossRef]
48. He, H.; Aogu, K.; Li, M.; Xu, J.; Sheng, W.; Jones, S.B.; González-Teruel, J.D.; Robinson, D.A.; Horton, R.; Bristow, K. A review of time domain reflectometry (TDR) applications in porous media. *Adv. Agron.* **2021**, *168*, 83–155.
49. JOHNSON, M.A.M.; PHANG, S.K.; WONG, W.; Chew, W.; Mun, H.; Hoon, Y. Distributed fiber optic sensing landslide monitoring—a comparative review. *J. Eng. Sci. Technol.* **2023**, *18*, 406–423.
50. Allil, R.C.; Lima, L.A.; Allil, A.S.; Werneck, M.M. FBG-based inclinometer for landslide monitoring in tailings dams. *IEEE Sens. J.* **2021**, *21*, 16670–16680. [CrossRef]
51. Xu, D.; Jiang, L.; Qin, Y.; Shen, H.; Ji, B. High-precision FBG-based sensor for soil settlement monitoring: A comparative study with magnetic settlement gauges and PIV technique. *Sens. Actuators A Phys.* **2024**, *366*, 114935. [CrossRef]
52. Chung, C.-C.; Lin, C.-P.; Ngui, Y.J.; Lin, W.-C.; Yang, C.-S. Improved technical guide from physical model tests for TDR landslide monitoring. *Eng. Geol.* **2022**, *296*, 106417. [CrossRef]
53. Ebrahim, K.M.; Gomaa, S.M.; Zayed, T.; Alfalah, G. Recent Phenomenal and Investigational Subsurface Landslide Monitoring Techniques: A Mixed Review. *Remote Sens.* **2024**, *16*, 385. [CrossRef]
54. Cataldo, A.; De Benedetto, E.; Masciullo, A.; Cannazza, G. A new measurement algorithm for TDR-based localization of large dielectric permittivity variations in long-distance cable systems. *Measurement* **2021**, *174*, 109066. [CrossRef]
55. Ye, X.; Zhu, H.H.; Wang, J.; Zhang, Q.; Shi, B.; Schenato, L.; Pasuto, A. Subsurface multi-physical monitoring of a reservoir landslide with the fiber-optic nerve system. *Geophys. Res. Lett.* **2022**, *49*, e2022GL098211. [CrossRef]

56. Wang, D.-Y.; Zhu, H.-H.; Wang, J.; Sun, Y.-J.; Schenato, L.; Pasuto, A.; Shi, B. Characterization of sliding surface deformation and stability evaluation of landslides with fiber-optic strain sensing nerves. *Eng. Geol.* **2023**, *314*, 107011. [CrossRef]
57. Ye, X.; Zhu, H.-H.; Cheng, G.; Pei, H.-F.; Shi, B.; Schenato, L.; Pasuto, A. Thermo-hydro-poro-mechanical responses of a reservoir-induced landslide tracked by high-resolution fiber optic sensing nerves. *J. Rock Mech. Geotech. Eng.* **2024**, *16*, 1018–1032. [CrossRef]
58. Ivanov, V.; Longoni, L.; Ferrario, M.; Brunero, M.; Arosio, D.; Papini, M. Applicability of an interferometric optical fibre sensor for shallow landslide monitoring—Experimental tests. *Eng. Geol.* **2021**, *288*, 106128. [CrossRef]
59. Ghazali, M.F.; Mohamad, H.; Nasir, M.Y.M.; Hamzh, A.; Abdullah, M.A.; Abd Aziz, N.F.; Thansirichaisree, P.; Zan, M.S.D. State-of-The-Art application and challenges of optical fibre distributed acoustic sensing in civil engineering. *Opt. Fiber Technol.* **2024**, *87*, 103911. [CrossRef]
60. Bordoni, M.; Vivaldi, V.; Boni, R.; Spanò, S.; Tararbra, M.; Lanteri, L.; Parnigoni, M.; Grossi, A.; Figini, S.; Meisina, C. A methodology for the analysis of continuous time-series of automatic inclinometers for slow-moving landslides monitoring in Piemonte region, northern Italy. *Nat. Hazards* **2023**, *115*, 1115–1142. [CrossRef]
61. Hu, M.; Ren, Y.; Xiao, L.; Liu, X.; Tan, Q.; Han, H.; Guo, J. Landslide monitoring based on GNSS and automated inclinometer systems and dynamic modeling using SPH: A case study in Qinghai, China. *Environ. Earth Sci.* **2024**, *83*, 170. [CrossRef]
62. Šegina, E.; Peternel, T.; Urbančič, T.; Realini, E.; Zupan, M.; Jež, J.; Caldera, S.; Gatti, A.; Tagliaferro, G.; Consoli, A. Monitoring surface displacement of a deep-seated landslide by a low-cost and near real-time GNSS system. *Remote Sens.* **2020**, *12*, 3375. [CrossRef]
63. Peternel, T.; Janža, M.; Šegina, E.; Bezak, N.; Maček, M. Recognition of landslide triggering mechanisms and dynamics using GNSS, UAV photogrammetry and in situ monitoring data. *Remote Sens.* **2022**, *14*, 3277. [CrossRef]
64. Fusco, F.; Bordoni, M.; Tufano, R.; Vivaldi, V.; Meisina, C.; Valentino, R.; Bittelli, M.; De Vita, P. Hydrological regimes in different slope environments and implications on rainfall thresholds triggering shallow landslides. *Nat. Hazards* **2022**, *114*, 907–939. [CrossRef]
65. Cui, Q.; Zhang, L.; Chen, X.; Cao, Z.; Wei, X.; Zhang, J.; Xu, J.; Liu, D.; Du, C. Quantitative risk assessment of landslides with direct simulation of pre-failure to post-failure behaviors. *Acta Geotech.* **2022**, *17*, 4497–4514. [CrossRef]
66. Hussain, Y.; Schlögel, R.; Innocenti, A.; Hamza, O.; Iannucci, R.; Martino, S.; Havenith, H.-B. Review on the geophysical and UAV-based methods applied to landslides. *Remote Sens.* **2022**, *14*, 4564. [CrossRef]
67. Imani, P.; Tian, G.; Hadiloo, S.; Abd El-Raouf, A. Application of combined electrical resistivity tomography (ERT) and seismic refraction tomography (SRT) methods to investigate Xiaoshan District landslide site: Hangzhou, China. *J. Appl. Geophys.* **2021**, *184*, 104236. [CrossRef]
68. Upadhyay, A.; Sharma, S. Mapping of alteration zones associated with uranium mineralization using joint analysis of quasi 3D ERT with gravity anomaly near Kutni, South Purulia Shear Zone, India. *J. Appl. Geophys.* **2020**, *179*, 104117. [CrossRef]
69. Lu, K.; Li, F.; Pan, J.; Li, K.; Li, Z.; Wang, P. Structural characterization and attempted displacement interpretation of the Baishuihe landslide using integrated geophysical methods. *Eng. Geol.* **2024**, *336*, 107568. [CrossRef]
70. Imani, P.; Abd EL-Raouf, A.; Tian, G. Landslide investigation using Seismic Refraction Tomography method: A review. *Ann. Geophys.* **2021**, *64*, SE657.
71. Xiaochun, L.; Xue, C.; Bobo, X.; Bin, T.; Xiaolong, T.; Zhigang, T. Bi-LSTM-GPR algorithms based on a high-density electrical method for inverting the moisture content of landslide. *Bull. Eng. Geol. Environ.* **2022**, *81*, 491. [CrossRef]
72. Deng, L.; Yuan, H.; Chen, J.; Zhang, M.; Su, G.; Pan, R.; Chen, Y.; Meng, X. Prefabricated acoustic emission array system for landslide monitoring. *Eng. Geol.* **2023**, *323*, 107185. [CrossRef]
73. Mohan, A.; Singh, A.K.; Kumar, B.; Dwivedi, R. Review on remote sensing methods for landslide detection using machine and deep learning. *Trans. Emerg. Telecommun. Technol.* **2021**, *32*, e3998. [CrossRef]
74. Liu, X.; Zhao, C.; Zhang, Q.; Yin, Y.; Lu, Z.; Samsonov, S.; Yang, C.; Wang, M.; Tomás, R. Three-dimensional and long-term landslide displacement estimation by fusing C-and L-band SAR observations: A case study in Gongjue County, Tibet, China. *Remote Sens. Environ.* **2021**, *267*, 112745. [CrossRef]
75. Tzouvaras, M.; Danezis, C.; Hadjimitsis, D.G. Differential SAR interferometry using Sentinel-1 imagery-limitations in monitoring fast moving landslides: The case study of Cyprus. *Geosciences* **2020**, *10*, 236. [CrossRef]
76. Mandlbürger, G. A review of active and passive optical methods in hydrography. *Int. Hydrogr. Rev.* **2022**, *28*, 8–52. [CrossRef]
77. Liang, S.; He, T.; Huang, J.; Jia, A.; Zhang, Y.; Cao, Y.; Chen, X.; Chen, X.; Cheng, J.; Jiang, B. Advances in high-resolution land surface satellite products: A comprehensive review of inversion algorithms, products and challenges. *Sci. Remote Sens.* **2024**, *10*, 100152. [CrossRef]
78. Strzabala, K.; Ćwiakała, P.; Puniach, E. Identification of Landslide Precursors for Early Warning of Hazards with Remote Sensing. *Remote Sens.* **2024**, *16*, 2781. [CrossRef]
79. Xie, M.; Zhao, W.; Ju, N.; He, C.; Huang, H.; Cui, Q. Landslide evolution assessment based on InSAR and real-time monitoring of a large reactivated landslide, Wenchuan, China. *Eng. Geol.* **2020**, *277*, 105781. [CrossRef]

80. Shiotani, T.; Ohtsu, M.; Ikeda, K. Detection and evaluation of AE waves due to rock deformation. *Constr. Build. Mater.* **2001**, *15*, 235–246. [CrossRef]
81. Li, M.; He, Y.; Meng, Z.; Wang, J.; Zou, X.; Hu, Y.; Zhao, Z. Acoustic emission-based experimental analysis of mechanical stress wave in IGBT device. *IEEE Sens. J.* **2020**, *20*, 6064–6074. [CrossRef]
82. Grosse, C.U.; Ohtsu, M.; Aggelis, D.G.; Shiotani, T. *Acoustic Emission Testing: Basics for Research–Applications in Engineering*; Springer Nature: Berlin/Heidelberg, Germany, 2021.
83. Ren, F.; Zhu, C.; He, M. Moment tensor analysis of acoustic emissions for cracking mechanisms during schist strain burst. *Rock Mech. Rock Eng.* **2020**, *53*, 153–170. [CrossRef]
84. Lin, W.; Mao, W.; Liu, A.; Koseki, J. Application of an acoustic emission source-tracing method to visualise shear banding in granular materials. *Géotechnique* **2021**, *71*, 925–936. [CrossRef]
85. Zhang, J.-Z.; Zhou, X.-P.; Du, Y.-H. Cracking behaviors and acoustic emission characteristics in brittle failure of flawed sandstone: A true triaxial experiment investigation. *Rock Mech. Rock Eng.* **2023**, *56*, 167–182. [CrossRef]
86. Komijani, M.; Gracie, R.; Yuan, Y. Simulation of fracture propagation induced acoustic emission in porous media. *Eng. Fract. Mech.* **2020**, *229*, 106950. [CrossRef]
87. Deng, L.; Yuan, H.; Chen, J.; Sun, Z.; Fu, M.; Wang, F.; Yan, S.; Li, K.; Yu, M.; Chen, T. Correlation between acoustic emission behaviour and dynamics model during three-stage deformation process of soil landslide. *Sensors* **2021**, *21*, 2373. [CrossRef]
88. He, Y.; Li, M.; Meng, Z.; Chen, S.; Huang, S.; Hu, Y.; Zou, X. An overview of acoustic emission inspection and monitoring technology in the key components of renewable energy systems. *Mech. Syst. Signal Process.* **2021**, *148*, 107146. [CrossRef]
89. Kim, H.; Choi, Y.; Park, D.; Lee, M.-K.; Lee, G.-J.; Kim, B.H.; Hong, J.-W. Numerical evaluation of acoustic emission (AE) sensors by lead-zirconate-titanate (PZT) geometric design. *Sens. Actuators A Phys.* **2024**, *366*, 115031. [CrossRef]
90. Fu, J.; He, T.; Liu, Z.; Bao, Y.; Liu, X. A novel waveguide rod with acoustic black hole for acoustic emission signal enhancement and its performance. *Ultrasonics* **2024**, *138*, 107260. [CrossRef]
91. Godin, N.; Reynaud, P.; Fantozzi, G. Challenges and limitations in the identification of acoustic emission signature of damage mechanisms in composites materials. *Appl. Sci.* **2018**, *8*, 1267. [CrossRef]
92. Li, S.; Feng, J.; Liu, Z.; Xu, B.; Wu, G. Spatial propagation characteristics of acoustic emission signals in parallel steel wire cables. *Measurement* **2024**, *226*, 114138. [CrossRef]
93. Willberry, J.O.; Papaelias, M.; Franklyn Fernando, G. Structural health monitoring using fibre optic acoustic emission sensors. *Sensors* **2020**, *20*, 6369. [CrossRef] [PubMed]
94. Prajna, K.; Mukhopadhyay, C. Fractional Fourier transform based adaptive filtering techniques for acoustic emission signal enhancement. *J. Nondestruct. Eval.* **2020**, *39*, 14. [CrossRef]
95. Li, D.; Wang, Y.; Yan, W.-J.; Ren, W.-X. Acoustic emission wave classification for rail crack monitoring based on synchrosqueezed wavelet transform and multi-branch convolutional neural network. *Struct. Health Monit.* **2021**, *20*, 1563–1582. [CrossRef]
96. Hou, D.; Qi, H.; Luo, H.; Wang, C.; Yang, J. Comparative study on the use of acoustic emission and vibration analyses for the bearing fault diagnosis of high-speed trains. *Struct. Health Monit.* **2022**, *21*, 1518–1540. [CrossRef]
97. Verstrynge, E.; Lacidogna, G.; Accornero, F.; Tomor, A. A review on acoustic emission monitoring for damage detection in masonry structures. *Constr. Build. Mater.* **2021**, *268*, 121089. [CrossRef]
98. Zhao, K.; Yang, D.; Gong, C.; Zhuo, Y.; Wang, X.; Zhong, W. Evaluation of internal microcrack evolution in red sandstone based on time–frequency domain characteristics of acoustic emission signals. *Constr. Build. Mater.* **2020**, *260*, 120435. [CrossRef]
99. Machorro-Lopez, J.M.; Hernandez-Figueroa, J.A.; Carrion-Viramontes, F.J.; Amezcua-Sanchez, J.P.; Valtierra-Rodriguez, M.; Crespo-Sanchez, S.E.; Yanez-Borjas, J.J.; Quintana-Rodriguez, J.A.; Martinez-Trujano, L.A. Analysis of acoustic emission signals processed with wavelet transform for structural damage detection in concrete beams. *Mathematics* **2023**, *11*, 719. [CrossRef]
100. Spasova, L.M.; Ojovan, M.I.; Gibb, F.G.F. Acoustic emission on melting/solidification of natural granite simulating very deep waste disposal. *Nucl. Eng. Des.* **2012**, *248*, 329–339. [CrossRef]
101. Hu, C.; Mei, F.; Hussain, W. Wavelet Energy Evolution Characteristics of Acoustic Emission Signal under True-Triaxial Loading during the Rockburst Test. *Appl. Sci.* **2022**, *12*, 7786. [CrossRef]
102. Zhao, T.; Zhang, P.; Xiao, Y.; Guo, W.; Zhang, Y.; Zhang, X. Master crack types and typical acoustic emission characteristics during rock failure. *Int. J. Coal Sci. Technol.* **2023**, *10*, 2. [CrossRef]
103. Du, K.; Li, X.; Tao, M.; Wang, S. Experimental study on acoustic emission (AE) characteristics and crack classification during rock fracture in several basic lab tests. *Int. J. Rock Mech. Min. Sci.* **2020**, *133*, 104411. [CrossRef]
104. Mao, W.; Towhata, I. Monitoring of single-particle fragmentation process under static loading using acoustic emission. *Appl. Acoust.* **2015**, *94*, 39–45. [CrossRef]
105. Xian, L.J.; Yah, C.S.; Tanaka, Y.; Hui, O.Y. Acoustic Emission Behavior of Tropical Residual Soil. *Int. J. Recent Technol. Eng.* **2020**, *8*, 1646–1655. [CrossRef]
106. Dagois-Bohy, S.; Ngo, S.; du Pont, S.C.; Douady, S. Laboratory singing sand avalanches. *Ultrasonics* **2010**, *50*, 127–132. [CrossRef]

107. Michlmayr, G.; Or, D. Mechanisms for acoustic emissions generation during granular shearing. *Granul. Matter* **2014**, *16*, 627–640. [CrossRef]
108. Siman-Tov, S.; Brodsky, E.E. Distinguishing between rheophysical regimes of fluid-saturated granular-flows using dilatancy and acoustic emission measurements. *Granul. Matter* **2021**, *23*, 44. [CrossRef]
109. Smith, A.; Dixon, N.; Meldrum, P.; Haslam, E.; Chambers, J. Acoustic emission monitoring of a soil slope: Comparisons with continuous deformation measurements. *Géotech. Lett.* **2014**, *4*, 255–261. [CrossRef]
110. Deng, L.; Yuan, H.; Chen, J.; Sun, Z.; Fu, M.; Zhou, Y.; Yan, S.; Zhang, Z.; Chen, T. Experimental investigation on progressive deformation of soil slope using acoustic emission monitoring. *Eng. Geol.* **2019**, *261*, 105295. [CrossRef]
111. Dixon, N.; Hill, R.; Kavanagh, J. Acoustic emission monitoring of slope instability: Development of an active waveguide system. *Proc. Inst. Civ. Eng.-Geotech. Eng.* **2003**, *156*, 83–95. [CrossRef]
112. Smith, A.; Dixon, N.; Fowmes, G. Monitoring buried pipe deformation using acoustic emission: Quantification of attenuation. *Int. J. Geotech. Eng.* **2017**, *11*, 418–430. [CrossRef]
113. Ono, K. Frequency dependence of receiving sensitivity of ultrasonic transducers and acoustic emission sensors. *Sensors* **2018**, *18*, 3861. [CrossRef]
114. Koerner, R.M.; McCabe, W.; Lord, A.E. Acoustic emission behavior and monitoring of soils. In *Acoustic Emissions in Geotechnical Engineering Practice*; ASTM International: West Conshohocken, PA, USA, 1981.
115. Lin, W.; Liu, A.; Mao, W.; Koseki, J. Acoustic emission behavior of granular soils with various ground conditions in drained triaxial compression tests. *Soils Found.* **2020**, *60*, 929–943. [CrossRef]
116. García-Ros, G.; Villalba-León, D.X.; Castro, E.; Sánchez-Pérez, J.F.; Valenzuela, J.; Conesa, M. Multivariate Statistical and Correlation Analysis between Acoustic and Geotechnical Variables in Soil Compression Tests Monitored by the Acoustic Emission Technique. *Mathematics* **2023**, *11*, 4085. [CrossRef]
117. Huang, S.; Lu, C.; Li, H.; He, J.; Wang, Q.; Gao, Z.; Yuan, P.; Li, Y. The attenuation mechanism and regular of the acoustic wave on propagation path in farmland soil. *Comput. Electron. Agric.* **2022**, *199*, 107138. [CrossRef]
118. Jurich, D.M.; Miller, R.J. Acoustic monitoring of landslides. *Transp. Res. Rec.* **1987**, *1119*, 30–38.
119. Hardy, H.R., Jr.; Taioli, F. Mechanical waveguides for use in AE/MS geotechnical applications. In *Progress in Acoustic Emission. IV*; JSNDI: Sao Paulo, Brazil, 1988; pp. 292–302.
120. Smith, A.; Heather-Smith, H.J.; Dixon, N.; Flint, J.A.; Pennie, D. Acoustic emission generated by granular soil–steel structure interaction. *Géotech. Lett.* **2020**, *10*, 119–127. [CrossRef]
121. Dixon, N.; Spriggs, M. Quantification of slope displacement rates using acoustic emission monitoring. *Can. Geotech. J.* **2007**, *44*, 966–976. [CrossRef]
122. Codeglia, D.; Dixon, N.; Fowmes, G.J.; Marcato, G. Analysis of acoustic emission patterns for monitoring of rock slope deformation mechanisms. *Eng. Geol.* **2017**, *219*, 21–31. [CrossRef]
123. Amitrano, D.; Grasso, J.R.; Senfaute, G. Seismic precursory patterns before a cliff collapse and critical point phenomena. *Geophys. Res. Lett.* **2005**, *32*, L0831. [CrossRef]
124. Got, J.-L.; Mourot, P.; Grangeon, J. Pre-failure behaviour of an unstable limestone cliff from displacement and seismic data. *Nat. Hazards Earth Syst. Sci.* **2010**, *10*, 819–829. [CrossRef]
125. Kramadibrata, S.; Simangunsong, G.M.; Matsui, K.; Shimada, H. Role of acoustic emission for solving rock engineering problems in Indonesian underground mining. *Rock Mech. Rock Eng.* **2011**, *44*, 281–289. [CrossRef]
126. Lockner, D.; Byerlee, J.D.; Kuksenko, V.; Ponomarev, A.; Sidorin, A. Quasi-static fault growth and shear fracture energy in granite. *Nature* **1991**, *350*, 39–42. [CrossRef]
127. Michlmayr, G.; Chalari, A.; Clarke, A.; Or, D. Fiber-optic high-resolution acoustic emission (AE) monitoring of slope failure. *Landslides* **2017**, *14*, 1139–1146. [CrossRef]
128. Langet, N.; Silverberg, F.M.J. Automated classification of seismic signals recorded on the Åknes rock slope, Western Norway, using a convolutional neural network. *Earth Surf. Dyn.* **2023**, *11*, 89–115. [CrossRef]
129. Chichibu, A.; Jo, K.; Nakamura, M.; Goto, T.; Kamata, M. Acoustic emission characteristics of unstable slopes. *J. Acoust. Emiss.* **1989**, *8*, 107–112.
130. Xue, D.; Lu, L.; Zhou, J.; Lu, L.; Liu, Y. Cluster modeling of the short-range correlation of acoustically emitted scattering signals. *Int. J. Coal Sci. Technol.* **2021**, *8*, 575–589. [CrossRef]
131. Spillmann, T.; Maurer, H.; Green, A.G.; Heincke, B.; Willenberg, H.; Husen, S. Microseismic investigation of an unstable mountain slope in the Swiss Alps. *J. Geophys. Res. Solid Earth* **2007**, *112*, B07301. [CrossRef]
132. Michlmayr, G.; Cohen, D.; Or, D. Sources and characteristics of acoustic emissions from mechanically stressed geologic granular media—A review. *Earth-Sci. Rev.* **2012**, *112*, 97–114. [CrossRef]
133. Hu, W.; Chen, L.; Shi, X.; Huang, Q. Enhancing Preamplifier Design for Acoustic Emission Detection Instrument. *J. Comput. Electron. Inf. Manag.* **2024**, *12*, 20–24. [CrossRef]

134. Acharya, A.; Kogure, T. Application of novel distributed fibre-optic sensing for slope deformation monitoring: A comprehensive review. *Int. J. Environ. Sci. Technol.* **2023**, *20*, 8217–8240. [CrossRef]
135. Polonelli, T.; Müller, H.; Kong, W.; Fischer, R.; Benini, L.; Magno, M. Aerosense: A self-sustainable and long-range bluetooth wireless sensor node for aerodynamic and aeroacoustic monitoring on wind turbines. *IEEE Sens. J.* **2022**, *23*, 715–723. [CrossRef]
136. Agbehadji, I.E.; Mabhaudhi, T.; Botai, J.; Masinde, M. A systematic review of existing early warning systems' challenges and opportunities in cloud computing early warning systems. *Climate* **2023**, *11*, 188. [CrossRef]
137. Nicolas, A.; Blöcher, G.; Kluge, C.; Li, Z.; Hofmann, H.; Pei, L.; Milsch, H.; Fortin, J.; Guéguen, Y. Pore pressure pulse migration in microcracked andesite recorded with fibre optic sensors. *Geomech. Energy Environ.* **2020**, *24*, 100183. [CrossRef]

Disclaimer/Publisher's Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Enhancing Grout Filling Quality Assessment in Precast Concrete Sleeve Connections through a Collaborative Sensing Approach

Bolin Jiang ^{1,2}, Shanshan Wu ^{2,3}, Qidong Xiong ^{2,4} and Yongsheng Yao ^{2,*}

¹ Department of Science and Technology, Chongqing Vocational Institute of Engineering, Chongqing 402260, China; bolinjiang@cqvie.edu.cn

² School of Civil Engineering, Chongqing Jiaotong University, Chongqing 400074, China; shanshanwu84@163.com (S.W.); xiongqd20@163.com (Q.X.)

³ School of Railways and Architecture, Chongqing Vocational College of Public Transportation, Chongqing 402247, China

⁴ Intelligent Construction Institute, Chongqing Jianzhu College, Chongqing 400072, China

* Correspondence: yaoyongsheng23@cqjtu.edu.cn

Abstract: This study presents a collaborative sensing approach that integrates the pre-embedded sensor method and the impact-echo technique to enhance the accuracy of grout filling quality assessment for precast concrete sleeve connections. The pre-embedded sensor method, which relies on vibration energy attenuation, enables continuous monitoring of the grout filling process; however, its accuracy is limited at low filling degrees, as vibration energy values remain constant at approximately 255 when the filling degree is below 70%. In contrast, the impact-echo technique, based on the principle of impact elastic wave propagation, demonstrates high accuracy in evaluating grout filling degrees across various levels, with reflected waveform amplitude increasing accordingly. This collaborative approach establishes a functional relationship between vibration energy values from the pre-embedded sensor method and grout filling degree, allowing for a comprehensive assessment of grout filling quality. In field demonstrations, the calculated grout filling degree values deviated by less than 5% from the set values. Practical guidelines for implementing the collaborative sensing approach are also provided. The method developed in this study offers a reliable solution for assessing grout filling quality in precast concrete sleeve connections, addressing the limitations of individual testing methods.

Keywords: precast concrete; sleeve connections; grout filling quality; collaborative sensing; pre-embedded sensor; impact-echo technique

1. Introduction

Precast concrete structures have emerged as a highly industrialized form of construction, offering a range of advantages, including environmental friendliness, low-carbon footprint, rapid installation, stability, and comparable mechanical performance to traditional cast-in-place concrete structures [1–3]. The key to the success of precast concrete structures lies in the seamless integration of the precast members into a monolithic system, which is primarily achieved through the connections between various structural elements, such as the beam-column and slab-column joints [4].

In the realm of precast concrete structures, the connections between different components play a crucial role in determining the overall structural performance [5,6]. These connection points are typically the locations where the highest stresses and deformations are concentrated, and their quality and integrity directly impact the safety, stability, and serviceability of the entire building [7]. Depending on the construction method, precast concrete connections can be broadly classified into two categories: wet connections and dry connections.

Wet connections, such as the grouted sleeve connection, have become the predominant choice in the majority of precast concrete structures due to their ability to create a continuous load transfer path and achieve a monolithic behavior [4,8]. In a grouted sleeve connection, the precast members are connected by inserting the reinforcing bars into a sleeve, which is then filled with a cementitious grout material [9,10]. The quality and completeness of the grout filling within the sleeve directly influence the structural integrity and load-bearing capacity of the connection [7,11–13].

Ensuring the quality of grouted sleeve connections is of paramount importance for the overall safety and stability of precast concrete structures. Deficiencies in the grout filling can lead to a reduction in the load-bearing capacity, compromising the structural integrity and potentially jeopardizing the entire building [14]. To address this critical aspect, a comprehensive quality control regime is essential, involving a range of testing methods and equipment to assess the quality of the grouted connections.

The quality inspection methods for grouted sleeve connections can be broadly categorized into two groups: destructive testing and non-destructive testing. Destructive testing, such as core sampling and pull-out tests, provides direct information about the grout properties and the integrity of the connection. However, these methods are invasive and can potentially compromise the structural integrity of the precast members. On the other hand, non-destructive testing techniques offer a more viable solution, as they can assess the grout quality without causing any physical damage to the structure [15–17].

Among the various non-destructive testing methods, the embedded sensor method and the impact-echo technique have gained widespread recognition in the assessment of grouted sleeve connections [18]. The embedded sensor method involves the installation of sensors within the grouting sleeve during the construction phase. After the grouting process is complete, the sensors collect data on the vibration energy, which can be used to evaluate the degree of grout filling and the quality of the connection [19]. The impact-echo technique, on the other hand, utilizes the principle of stress wave propagation to detect defects and density variations within the grouted sleeve [20,21]. By analyzing the reflected stress waves, the technique can provide insights into the integrity and compactness of the grout filling. Each of these non-destructive testing methods has its own strengths and limitations [22]. The embedded sensor method is particularly well-suited for in-situ applications, where the sleeves are already embedded within the precast concrete elements [19,23]. In such scenarios, the lack of access to the exposed steel reinforcement ends prevents the direct application of the impact-echo technique. Conversely, the impact-echo method can provide more detailed information about the internal condition of the grouted sleeve, including the presence of voids or defects, but it requires access to the exposed steel reinforcement ends for the excitation of the stress waves [24].

To overcome the limitations of the individual testing methods and leverage their respective advantages, a collaborative sensing approach can be developed by exploring the correlation between the vibration energy values obtained from the embedded sensor method and the density information derived from the impact-echo technique [25,26]. By establishing a relationship between these two parameters, it becomes possible to enhance the accuracy and reliability of the grouting quality assessment for sleeve connections, particularly in situations where the impact-echo technique cannot be directly applied.

This study aimed to investigate the relationship between the vibration energy values measured by the embedded sensor method and the density information obtained from the impact-echo technique, and to establish a method for the collaborative sensing of grouting filling degree in sleeve connections. The research workflow is presented in Figure 1. The performance of the embedded sensor method in assessing the grout filling degree of sleeve connections under varying levels of grout filling was analyzed. The effectiveness of the impact-echo technique in determining the grout filling degree of sleeve connections was evaluated, and a correlation between the detected density and the actual grout filling degree was established. A collaborative sensing approach was developed by exploring the relationship between the vibration energy values from the embedded

sensor method and the density information from the impact-echo technique to enhance the accuracy of grouting quality assessment for sleeve connections. Finally, the proposed collaborative sensing approach was validated through field demonstration and provided practical guidelines for its implementation in real-world precast concrete construction projects.

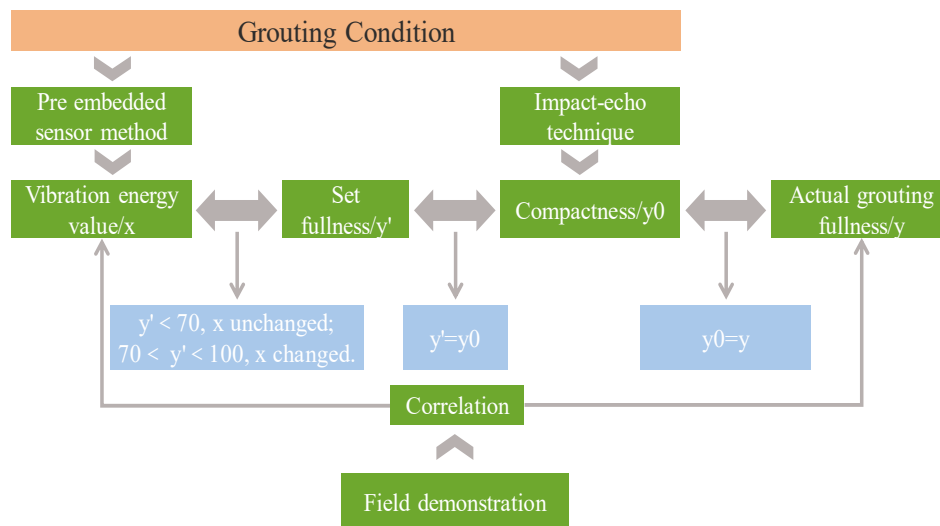


Figure 1. Research flowchart of this study.

The structure of this paper is organized as follows. Section 2 presents the materials and methods used in the study, including the detailed procedures for the embedded sensor method and the impact-echo technique, as well as the design of the experimental models. Section 3 discusses the results obtained from the laboratory experiments and the field demonstration, and analyzes the correlation between the vibration energy values and the grout filling degree. Finally, Section 4 summarizes the key findings of the study and provides recommendations for future research and practical applications.

The research presented in this paper contributes to the body of knowledge on the quality assessment of grouted sleeve connections in precast concrete structures. By developing a collaborative sensing approach that combines the strengths of the embedded sensor method and the impact-echo technique, the study offers a more comprehensive and reliable solution for evaluating the grout filling degree, which is critical for ensuring the structural integrity and safety of precast concrete buildings. The findings of this research can be valuable for practitioners, researchers, and policymakers in the precast concrete industry, helping to improve the quality control and construction practices for precast concrete structures.

2. Materials and Methods

2.1. Embedded Sensor Method

The embedded sensor method utilizes damped sensors to measure the vibration energy values. In this study, the ZBL-G1000 detector and SDA1003 damping vibration sensor from the Beijing ZBL Science and technology Co., Ltd. (Beijing, China) were used to detect the fullness of grouting. The principle is based on the damped vibration amplitude attenuation, wherein the sensor vibrates under the applied signal and experiences the effects of friction and medium resistance, causing the amplitude to decay over time. By analyzing the different vibration amplitude decay patterns, the surrounding medium environment of the sensor can be inferred, and the grout filling degree within the sleeve can be assessed.

According to the Chinese technical specification for inspection of sleeve grouting quality of precast concrete structures (T/CECS 683-2020) [27], the grout filling degree can

be evaluated based on the following criteria: when the vibration energy value is more than 100, the grout filling is considered complete; when the vibration energy value is between 100 and 150, the grout filling is essentially complete; and when the vibration energy value is >150 , the grout filling is incomplete.

The testing procedure for the embedded sensor method is illustrated in Figure 2. Figure 2a includes (1) weighing grouting material, (2) mixing grouting material, (3) sealing the lower end of the sleeve, (4) grouting, (5) testing, and (6) data acquisition. First, the damped sensor is inserted through the grout outlet, ensuring that the sensor reaches the bottom of the sleeve and is tightly sealed with a rubber plug. After the grouting is completed, the signal cable is connected to the instrument's sensor interface at one end, and the two clips are attached to the signal lines of the pre-embedded sensor within the tested sleeve. By clicking the sampling button or the sampling key on the main interface, the system automatically starts continuous sampling, and the waveform area displays the collected dynamic waveform. Upon completion of the sampling, the data are stored, and the next sleeve can be tested. This process is repeated until all the sleeves have been tested, and then the stop button is clicked.

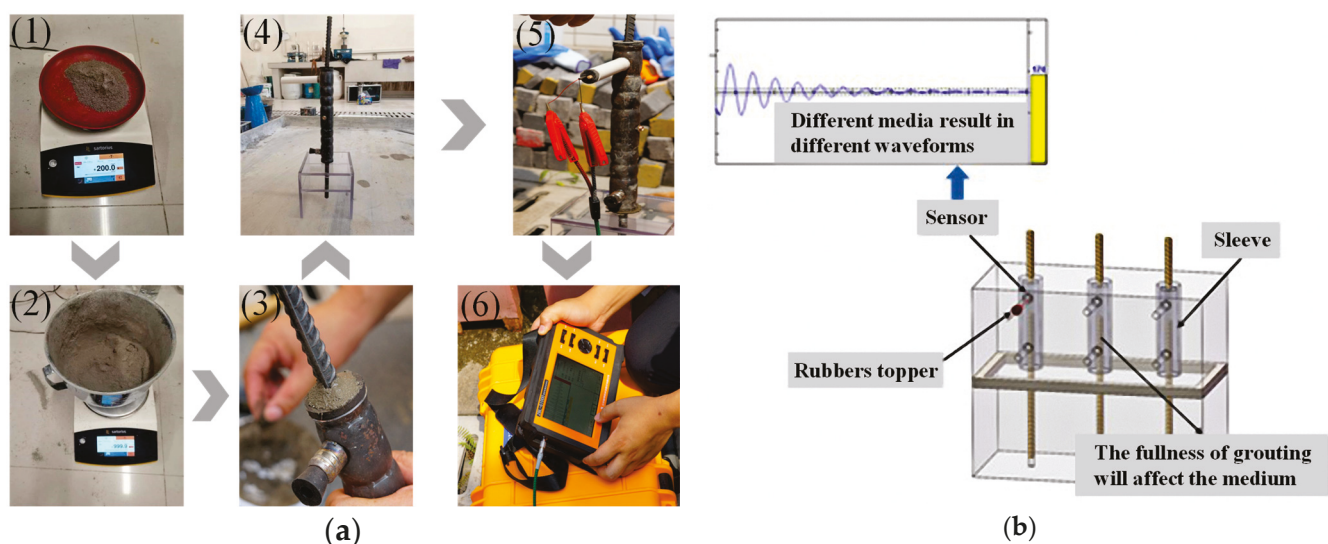


Figure 2. Process and schematic diagram of embedded sensor method testing: (a) testing flow, (b) schematic diagram.

2.2. Impact-Echo Technique

The impact-echo technique utilizes the principle of impact elastic wave propagation for testing. The test setup used in this study is shown in Figure 3. The detection instrument is the SRB-MATS-B multi-function rock anchor tester manufactured by Sichuan Sheng-Tuo (Chengdu, China), which is based on the impact elastic wave principle. The testing procedure involves using an impact hammer to strike the exposed reinforcement end at the center of the sleeve, generating an elastic wave. The sensor then records the elastic wave data, and through the analysis of the properties of the excitation and received signals (wave velocity, amplitude, and dominant frequency), the grout filling density within the sleeve can be qualitatively evaluated.

The testing procedure for the impact-echo technique is illustrated in Figure 4. The test requires the collaboration of two personnel. First, the sleeve is positioned on the fixture of the test bench to prevent the test component from shaking under external force during the test. A coupling agent is applied to the exposed reinforcement end of the sleeve, ensuring that the entire surface is not covered, leaving a portion exposed for manual excitation. The detection instrument is the SRB-MATS-B multi-function rock anchor tester manufactured by Sichuan Sheng-Tuo, which is based on the impact elastic wave principle. The detection instrument is composed of a signal receiver and a main instrument. The

signal receiver (with a magnetic base) is then attached to the coupling agent surface, and the receiver is connected to the main instrument. One of the personnel then gently strikes the reinforcement end with the impact hammer, while the other personnel monitors the data displayed on the instrument and adjusts the excitation force to obtain the optimal waveform. In this experiment, the optimal waveform is obtained with a specific quantification of the impact force measured at 12.15 N, corresponding to the impact force of a 50 g steel ball falling from a height of 30 cm. The distance between the excitation position and the sensor should be maintained at 0.02~0.03 m, and the instrument's sampling frequency should be set to 250 kHz. To ensure proper adhesion between the sensor and the contact element, the surface of the contact element should be polished prior to testing. The signal receiver is equipped with a magnetic base to prevent experimental errors caused by poor connection with the contact element.

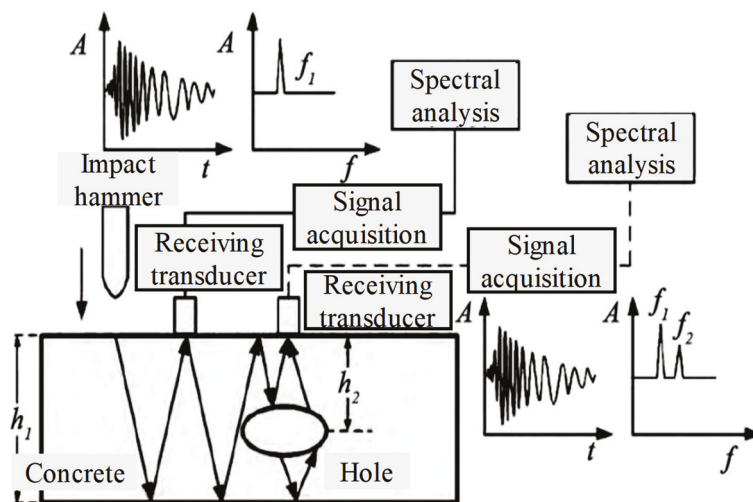


Figure 3. Impact-echo technique.

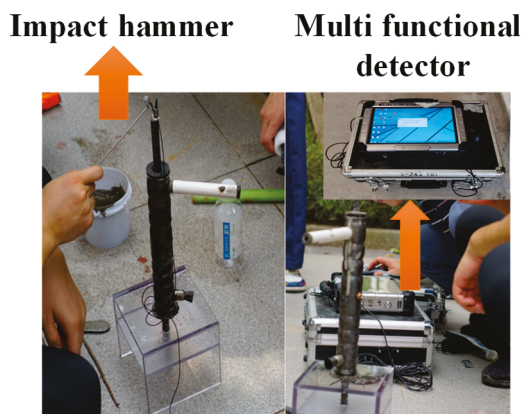


Figure 4. Schematic diagram of impact echo technique testing.

2.3. Experimental Model

The experimental model used in this study is shown in Figure 5, and it is composed of reinforcing steel, connection sleeves, cementitious grout, sealant, and a base. The red marker is the Grout stopper for Grout outlet. For the experiments, sleeves with different grout filling degrees were set up and used for testing. The grout filling degree is defined as the ratio of the actual grout volume within the sleeve to the maximum grout volume that can fill the sleeve completely, expressed as a percentage.

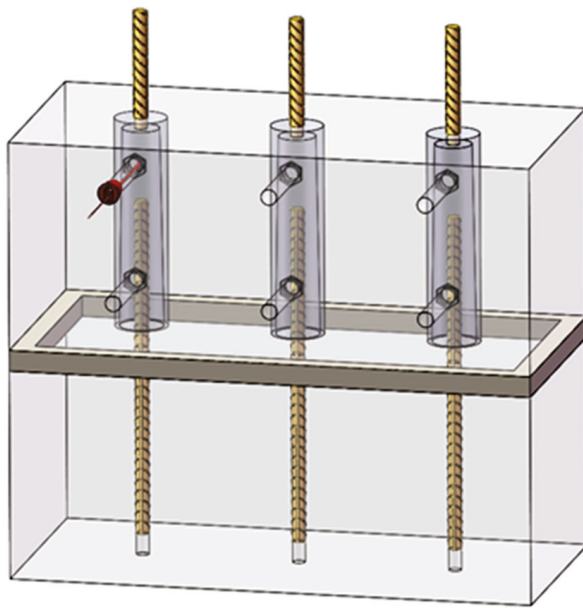


Figure 5. Experimental model for grouting quality of sleeve connection nodes.

Based on engineering experience, the grout filling degree of the sleeve is not only determined by the grout volume, but also influenced by the grout viscosity, grouting speed, sleeve diameter, and sleeve wall thickness. In this study, by referring to existing research and combining it with engineering practice, sleeve connection specimens with grout filling degrees of 10%, 20%, 30%, 40%, 50%, 60%, 70%, 80%, 90%, and 100% were fabricated in the laboratory to simulate the various grout filling scenarios [26,28].

2.4. Field Demonstration

The field demonstration of the proposed testing methods was conducted on a precast shear wall structure. The schematic diagram of the field demonstration is shown in Figure 6. The precast shear wall contains multiple sets of sleeve connections, with known sleeve configurations and volumes. The steps of the field demonstration are as follows:

- (1) The sleeves were first flushed with high-pressure water to remove any residual debris;
- (2) The grout and bedding mortar were prepared, and the bedding mortar was used to seal the bottom of the sleeves;
- (3) The damped vibration sensors were installed at the grout outlets, and the grout filling degree testing instrument was used to ensure the proper functioning of the sensors;
- (4) The grout was then pumped into each set of sleeves using a high-pressure grouting machine;
- (5) After the grouting was completed, the grout filling degree testing instrument was used to measure the waveform and vibration energy values of the grouted sleeves.



Figure 6. Schematic diagram of field demonstration.

3. Results and Discussion

3.1. Pre-Embedded Sensor Method

The experimental results obtained using the pre-embedded sensor method for different grout filling degrees are shown in Figure 7. The analysis of the results reveals the following:

From the measured vibration energy values and waveform plots, it can be observed that as the set grout filling degree increases, the vibration energy values initially remain unchanged and then decrease. Similarly, the number of waveforms gradually decreases and the waveform amplitude becomes smaller. When the set grout filling degree is 100%, the vibration energy waveform trends towards a straight line, which is consistent with the current specification for evaluating the grout filling degree based on the vibration energy values.

When the set grout filling degree is less than 70%, the vibration energy values remain unchanged, essentially maintaining a value of 255, and do not accurately reflect the changes in the grout filling degree. This is due to the fact that in the pre-embedded sensor method, the core testing component, the damped vibration sensor, is exposed to the air and not completely covered by the grout. This indicates a limitation in using the vibration energy values from the pre-embedded sensor method to assess the grout filling degree.

When the set grout filling degree is between 70% and 100%, the vibration energy values decrease as the grout filling degree increases. This is because, in this range, the grout covers the damped vibration sensor first, and then a portion of the grout recedes, causing the measured vibration energy values to change accordingly. This suggests a correlation between the vibration energy values and the grout filling degree in this range. It is worth noting that there is no linear relationship between grouting fullness and vibration energy value. When the set grout filling degree is between 70% and 100%, the relationship

between grouting fullness and vibration energy values in this study can be represented by Equation (1).

$$y = 0.0725x^2 - 14.175x + 810.75 \quad (1)$$

where y represents the vibration energy values and x represents the grouting fullness.

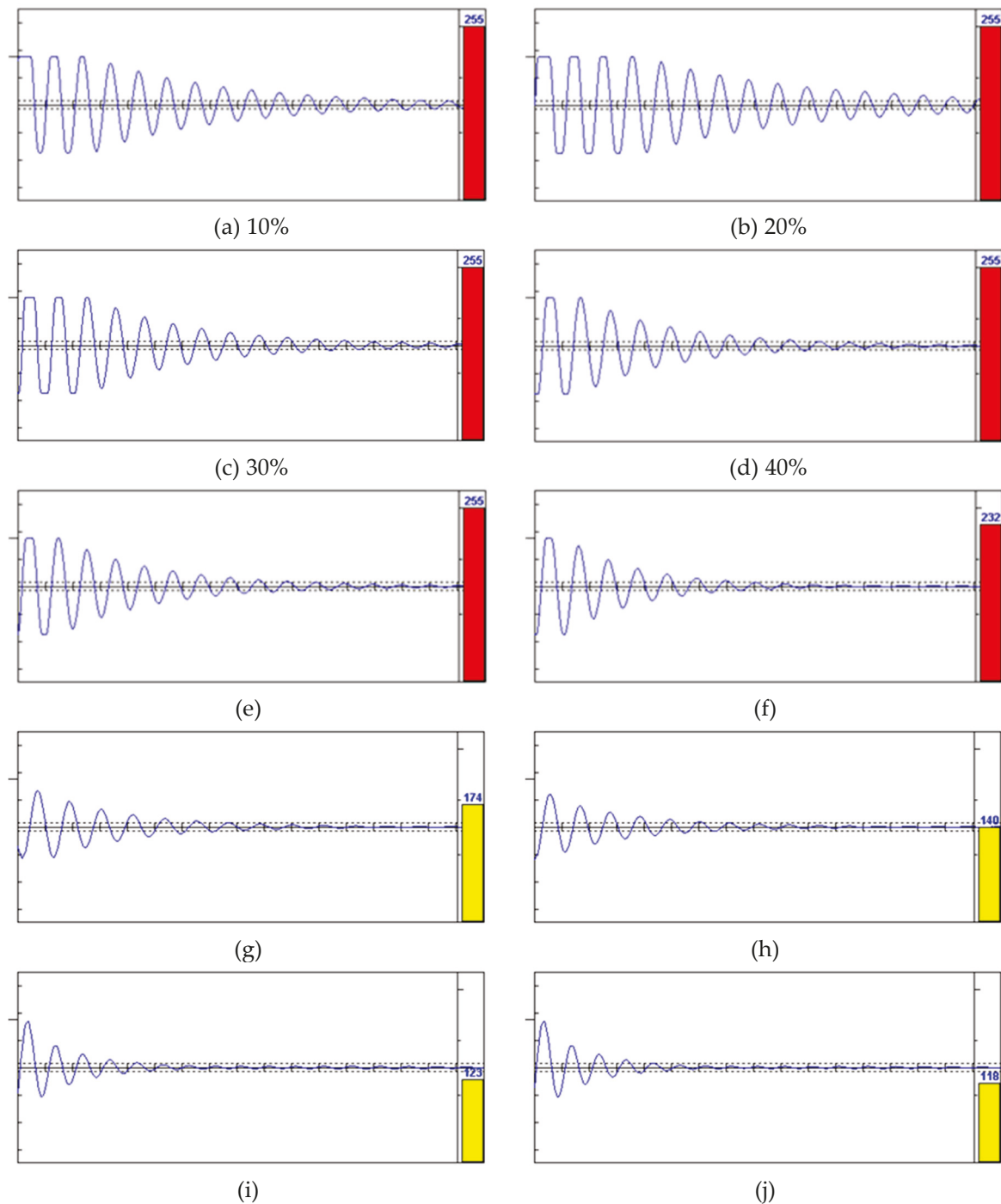


Figure 7. Test results of pre-embedded sensors under different grouting fullness: (a) 10%, (b) 20%, (c) 30%, (d) 40%, (e) 50%, (f) 60%, (g) 70%, (h) 80%, (i) 90%, (j) 100%.

3.2. Test Results of Impact-Echo Technique

The test results of the impact-echo technique for different grout filling degrees are shown in Figure 8. To ensure the accuracy of the experimental data, each set of grout filling degree was tested five times.

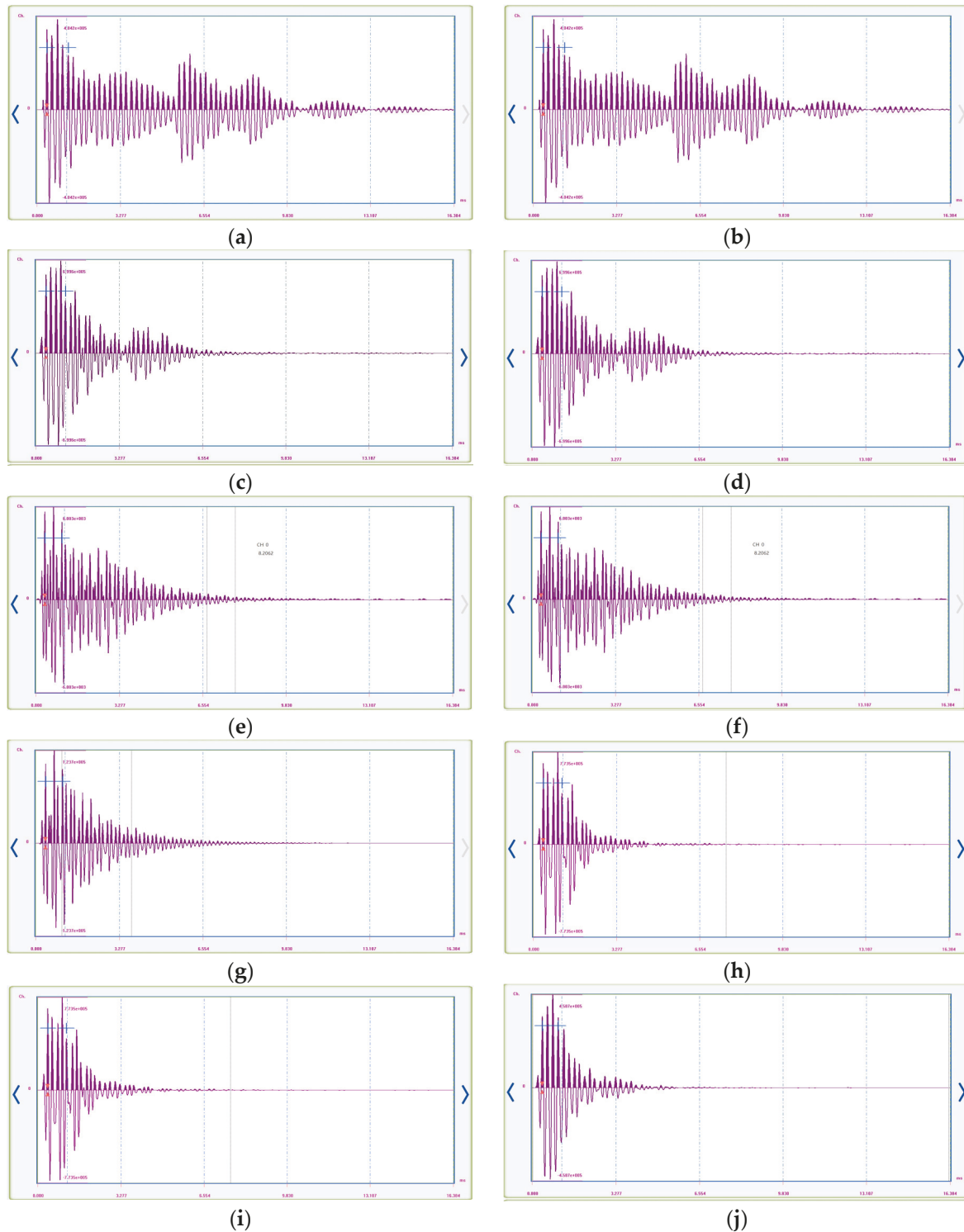


Figure 8. Test results of impact-echo technique under different grouting fullness: (a) 10%, (b) 20%, (c) 30%, (d) 40%, (e) 50%, (f) 60%, (g) 70%, (h) 80%, (i) 90%, (j) 100%.

The analysis of the results indicates that the reflected waveforms differ for different grout filling degrees. As the grout filling degree increases, the reflected waveforms received by the impact-echo detection instrument become stronger.

The experimental data obtained using the impact-echo technique are summarized in Table 1. The analysis of the data shows that the standard deviations of the measured filling degree results are relatively small (<2.5), indicating the stability of the test data and the absence of significant errors. Additionally, the percentage deviations of the test data are within 5%, which is an acceptable range, demonstrating that the measured data can accurately reflect the actual grout filling quality. This suggests that the impact-echo technique can reliably evaluate the grout filling degree in sleeve connections.

Table 1. Test results of impact-echo technique.

Fullness Setting Value/%	Fullness Detection Value/%	Average Value/%	Standard Deviation	Deviation/%
10	10	9.6	0.80	4.00
	9			
	9			
	9			
	11			
20	19	19.6	1.36	2.00
	20			
	22			
	18			
	19			
30	28	29.4	1.50	2.00
	29			
	32			
	30			
	28			
40	39	39.4	1.02	1.50
	39			
	38			
	40			
	41			
50	50	49.4	1.02	1.20
	49			
	51			
	49			
	48			
60	55	57.4	2.06	4.33
	56			
	61			
	57			
	58			
70	69	69.6	1.85	0.57
	70			
	73			
	68			
	68			
80	78	77.8	2.14	2.75
	75			
	76			
	81			
	79			

Table 1. Cont.

Fullness Setting Value/%	Fullness Detection Value/%	Average Value/%	Standard Deviation	Deviation/%
90	87	89	1.41	1.11
	90			
	89			
	88			
	91			
100	95	96.4	1.02	3.60
	97			
	96			
	98			
	96			

Note: Deviation = ((Detection value – Setting value)/Setting value) × 100%.

The impact-echo technique uses a multi-function rock anchor tester that is based on the principle of impact elastic wave propagation. When the stress wave encounters an interface with different acoustic impedance, it will be reflected, and the resonance phenomenon will cause changes in the surface displacement of the concrete. By placing receivers around the excitation point, the displacement signals can be collected and analyzed using the fast Fourier transform (FFT) to evaluate the internal defects and density of the structure [29–31].

The impact-echo technique has the advantages of high energy, strong penetration, convenient operation, and fast testing speed, making it particularly suitable for precast structures with single-sided access [32]. It also demonstrates a high accuracy in detecting the grout filling degree of both large and small diameter corrugated sleeves. In the field cases, under specific conditions, the impact-echo technique's results for the grout filling degree of large diameter corrugated sleeves were found to be consistent with the core sampling results.

3.3. Correlation Analysis

The analysis of the results from Sections 3.1 and 3.2 reveals that the vibration energy values obtained from the pre-embedded sensor method can reflect the grout filling quality within a certain range, while the density values measured by the impact-echo technique are highly correlated with the set grout filling degree.

Under field construction conditions, since the sleeves are pre-embedded in the concrete members, it is not possible to directly use the impact-echo technique for grout filling degree detection, as the excitation point can only be placed on the surface of the concrete member, and not on the exposed reinforcement ends. Therefore, this section explores the correlation between the grout filling degree and the vibration energy values obtained from the pre-embedded sensor method, which can help directly obtain the grout filling degree in the field construction environment using the pre-embedded sensor method.

To investigate the correlation between the vibration energy values obtained from the pre-embedded sensor method and the set grout filling degree, and to better reflect the relationship between the two, this study added three additional sets of grout filling degree experiments based on the pre-embedded sensor method test data, and the compiled test data are summarized in Table 2.

The fitting analysis of the data in Table 2 shows that the vibration energy values obtained from the pre-embedded sensors and the fullness values obtained from the impact-echo technique follow an exponential relationship, and the function relationship can be expressed as:

$$y' = 5978.5 \times \exp(-x/22.4) + 67.8 \quad (2)$$

where x is the vibration energy value ranging from 70% to 100%, and when $x < 70\%$, $y' = 255$.

As shown in Figure 9, the fitting curve has an R^2 value of 0.995, indicating a high degree of fit, which can be used to express the relationship between the vibration energy value and the set grout filling degree.

Table 2. Test results of pre-embedded sensors and impact-echo technique.

Pre-Embedded Sensors Vibration Energy Value (x)	Impact-Echo Technique Fullness Detection Value (y')/%
255	<70
174	70
152	75
140	80
131	85
125	90
120	95
118	100

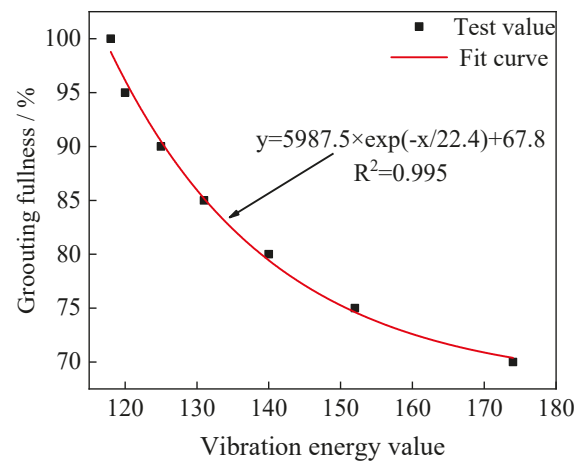


Figure 9. Fitting analysis of vibration energy value and grouting fullness.

Furthermore, since the density values measured by the impact-echo technique are essentially the same as the grout filling degree within the sleeve, i.e., $y' = y$, the function relationship derived in this section between the vibration energy values from the pre-embedded sensor method and the fullness values from the impact-echo technique can be used to represent the relationship between the vibration energy values and the grout filling degree within the sleeve.

3.4. Test Results of the Field Demonstration

To verify the accuracy of the function relationship between the vibration energy values and the grout filling degree, as fitted in Figure 9, for field experiments, this section conducted field tests using the pre-embedded sensor method to measure the grout filling degree after grouting. The precast shear wall used in the experiment contained multiple sets of sleeve connections. According to the design information of the shear wall, the internal structure and volume of the sleeves were known. The field experiment set the grout filling degrees at 60%, 65%, 70%, 75%, 80%, 85%, 90%, 95%, and 100%, a total of nine test conditions.

To avoid deviations in the grout filling degree test results for each set of sleeve connections due to factors such as internal structure and residual debris, the grout filling degree detection instrument was used to test each set three times, and the average value was taken.

After the experiment, the grout filling degree was calculated based on the test results and the function relationship in Equation (1), using the vibration energy values. The set grout filling degree and the experimentally calculated values are shown in Figure 10.

The analysis of Figure 10 shows that in the field experiment, the grout filling degree calculated from the vibration energy values measured using the pre-embedded sensor method, through the fitted function in Equation (1), has a small deviation from the set grout filling degree and a high reliability. This also indicates that Equation (1) can be used to represent the relationship between the vibration energy values and the grout filling degree within the sleeve, i.e., the grout filling degree can be directly evaluated through the pre-embedded sensor method.

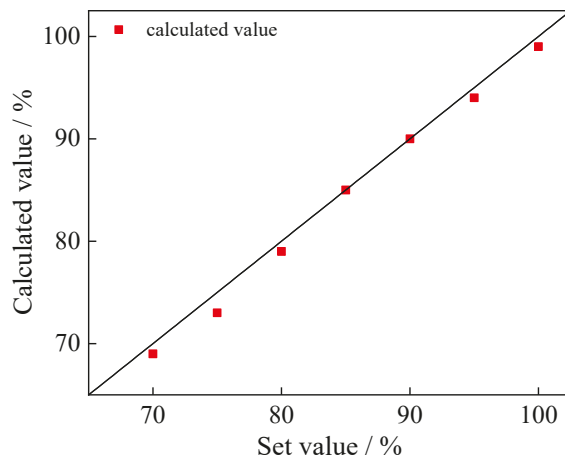


Figure 10. Deviation between set value and calculated value.

3.5. Comparison of Testing Methods

The pre-embedded sensor method and the impact-echo technique have their respective advantages and limitations in evaluating the grout filling degree of sleeve connections, as summarized in Table 3.

Table 3. Comparison of pre-embedded sensor method and impact-echo technique.

Characteristics	Pre-Embedded Sensor Method	Impact-Echo Technique
Principle	Vibration energy attenuation	Impact elastic wave propagation
Evaluation target	Vibration energy value	Density/Impedance
Applicability	Embedded sleeve	Exposed reinforcement end
Accessibility	Convenient, no special access required	Requires access to reinforcement end
Test duration	Short, continuous monitoring	Relatively longer, point-by-point testing
Accuracy	Limited at low filling degrees	High accuracy across filling degrees
Limitations	Air exposure of sensor, limited range	Requires access to reinforcement end

The pre-embedded sensor method has the advantage of convenient and continuous monitoring, as the sensors are pre-embedded in the sleeve and do not require special access for testing [33,34]. However, the method has limited accuracy in evaluating the grout filling degree, particularly when the filling degree is low, due to the air exposure of the sensor.

The impact-echo technique, on the other hand, demonstrates high accuracy in assessing the grout filling degree across different filling levels, as it is based on the principle of impact elastic wave propagation and can effectively detect the internal density/impedance changes [35–37]. However, the technique requires access to the exposed reinforcement ends for excitation, which may not be feasible in some field conditions.

To leverage the strengths of both testing methods and improve the overall accuracy of grout filling quality assessment, a collaborative sensing approach has been developed, as discussed in the following section.

3.6. Practical Guidelines for Implementation

Based on the findings from the laboratory experiments and the field demonstration, the following practical guidelines are provided for the implementation of the collaborative sensing approach in precast concrete construction projects:

(1) Pre-installation of embedded sensors:

During the prefabrication stage, install the damped vibration sensors at the grout outlets of the sleeve connections, ensuring that the sensors are properly sealed and integrated into the sleeve assembly.

Ensure the proper functioning of the pre-embedded sensors and the compatibility with the grout filling degree testing instrument. The grouting fullness detector is required to meet an amplitude linearity specification of within ± 1.0 dB for every 10.0 dB, with a frequency bandwidth between 10 kHz and 100 kHz. A damping vibration sensor should be utilized, ensuring that the diameter of the core component at the end does not exceed 10.0 mm, and the diameter of the steel wire connected to the core component is maintained between 2.0 mm and 3.0 mm. Additionally, the sensor and rubber plug should be designed as an integrated unit, with the diameter of the hole through which the steel wire passes matching the diameter of the steel wire, and the diameter of the exhaust hole being no less than 3.0 mm;

(2) Grout filling and testing procedure:

Flush the sleeves with high-pressure water to remove any residual debris before grouting.

Prepare the grout and bedding mortar, and use the bedding mortar to seal the bottom of the sleeves.

Connect the pre-embedded sensor signal cables to the grout filling degree testing instrument, and initiate the continuous monitoring of the vibration energy values during the grouting process.

After the grouting is completed, record the final vibration energy values for each sleeve connection;

(3) Collaborative data analysis:

Use the function relationship in Equation (1) to convert the vibration energy values from the pre-embedded sensor method into the corresponding grout filling degree values.

If access to the exposed reinforcement ends is available, conduct the impact-echo technique testing to obtain the density information and cross-validate the grout filling degree results.

Integrate the vibration energy-based grout filling degree and the impact-echo-based density information to establish the collaborative assessment of the grout filling quality;

(4) Quality assurance and decision-making:

Compare the collaborative assessment results with the specified grout filling degree requirements, as per the relevant technical specifications.

Identify any sleeve connections with inadequate grout filling and prioritize them for remedial actions, such as additional grouting or replacement.

Implement the collaborative sensing approach as a standard quality control procedure for precast concrete sleeve connections, ensuring consistent and reliable grout filling quality.

By following these practical guidelines, construction teams can effectively implement the collaborative sensing approach to enhance the assessment of grout filling quality in precast concrete sleeve connections, leading to improved structural integrity and long-term performance of the precast concrete structures.

4. Conclusions

This study developed a collaborative sensing approach that combines the pre-embedded sensor method and the impact-echo technique to enhance the accuracy of grout filling qual-

ity assessment for precast concrete sleeve connections. The key findings and conclusions drawn from the research are as follows:

- (1) The pre-embedded sensor method, based on the principle of vibration energy attenuation, can provide continuous monitoring of the grout filling process. However, its accuracy is limited at low filling degrees due to the exposure of the sensor to air;
- (2) The impact-echo technique, utilizing the principle of impact elastic wave propagation, demonstrates a high degree of accuracy in evaluating the grout filling degree across different filling levels. It can reliably detect the internal density and impedance changes within the sleeve connections;
- (3) The collaborative sensing approach establishes a functional relationship between the vibration energy values obtained from the pre-embedded sensor method and the grout filling degree, thereby leveraging the strengths of both testing methods. This approach can provide a comprehensive and accurate assessment of the grout filling quality, even in field conditions where direct access to the reinforcement ends is not feasible;
- (4) The field demonstration has validated the effectiveness of the collaborative sensing approach, with the calculated grout filling degree values closely matching the set values. This indicates the practical applicability of the proposed method in real-world precast concrete construction projects;
- (5) Practical guidelines have been provided for the implementation of the collaborative sensing approach, including the pre-installation of embedded sensors, the grout filling and testing procedure, the collaborative data analysis, and the quality assurance and decision-making process. These guidelines can assist construction teams in the effective implementation of this approach in their projects.

The collaborative sensing approach developed in this study offers a reliable and comprehensive solution for assessing the grout filling quality in precast concrete sleeve connections, contributing to the overall structural integrity and performance of precast concrete structures. By integrating the strengths of both the pre-embedded sensor method and the impact-echo technique, this approach can overcome the limitations of each individual method and provide a robust and accurate evaluation of the grout filling quality.

Future research directions may include the investigation of advanced data analysis techniques, the exploration of wireless sensor integration, and the development of automated monitoring and decision-making frameworks to further streamline the implementation of the collaborative sensing approach in the construction industry.

Author Contributions: Conceptualization, Y.Y. and B.J.; methodology, S.W.; software, Q.X.; validation, Y.Y. and B.J.; formal analysis, S.W.; investigation, B.J. and S.W.; resources, Q.X.; data curation, Q.X.; writing—original draft preparation, B.J.; writing—review and editing, B.J. and Y.Y.; project administration, S.W.; funding acquisition, Q.X. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the Science and Technology Research Project of Chongqing Education Commission (Grant numbers: KJQN202205802, KJQN202305802).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The raw data supporting the conclusions of this paper will be made available by the authors by request.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Yee, A.A.; Eng, P.H.D. Social and environmental benefits of precast concrete technology. *PCI J.* **2001**, *46*, 14–19. [CrossRef]
2. Yee, A.A.; Eng, P. Structural and economic benefits of precast/prestressed concrete construction. *PCI J.* **2001**, *46*, 34–43. [CrossRef]
3. Kurama, Y.C.; Sritharan, S.; Fleischman, R.B.; Restrepo, J.I.; Henry, R.S.; Cleland, N.M.; Ghosh, S.K.; Bonelli, P. Seismic-Resistant Precast Concrete Structures: State of the Art. *J. Struct. Eng.* **2018**, *144*, 03118001. [CrossRef]

4. Sun, Y.L.; Sun, Z.Y.; Yao, L.Z.; Wei, Y.; Wu, G. Bond performance between SFCBs and grouted sleeves for precast concrete structures. *Adv. Struct. Eng.* **2021**, *24*, 2857–2869. [CrossRef]
5. Kataoka, M.N.; Ferreira, M.A.; El Debs, A. Study on the behavior of beam-column connection in precast concrete structure. *Comput. Concr.* **2015**, *16*, 163–178. [CrossRef]
6. Yu, Q.; Sun, J.; Xu, Z.; Li, L.; Zhang, Z.; Yu, S. Mechanical analysis of grouted sleeve lapping connector. *Appl. Sci.* **2019**, *9*, 4867. [CrossRef]
7. Li, X.M.; Xiao, S.; Gao, R.D.; Harries, K.A.; Wang, Z.L.; Xu, Q.F. Effects of grout sleeve defects and their repair on the seismic performance of precast concrete frame structures. *Eng. Struct.* **2021**, *242*, 112619. [CrossRef]
8. Belleri, A.; Riva, P. Seismic performance and retrofit of precast concrete grouted sleeve connections. *PCI J.* **2012**, *57*, 97–109. [CrossRef]
9. Alshaikh, I.M.; Abadel, A.A.; Alrubaidi, M. Precast RC structures' progressive collapse resistance: Current knowledge and future requirements. *Structures* **2022**, *37*, 338–352. [CrossRef]
10. Tullini, N.; Minghini, F. Grouted sleeve connections used in precast reinforced concrete construction—Experimental investigation of a column-to-column joint. *Eng. Struct.* **2016**, *127*, 784–803. [CrossRef]
11. Tengfei, X.; Quanwen, L.; Renda, Z.; Ding, J.; Zhan, Y. On the early-age bond-slip behavior of an eccentric bar embedded in a grouted sleeve. *Eng. Struct.* **2019**, *190*, 160–170.
12. Zheng, G.; Kuang, Z.; Xiao, J.; Pan, Z. Mechanical performance for defective and repaired grouted sleeve connections under uniaxial and cyclic loadings. *Constr. Build. Mater.* **2020**, *233*, 117233. [CrossRef]
13. Xu, F.; Wang, K.; Wang, S.; Li, W.; Liu, W.; Du, D. Experimental bond behavior of deformed rebars in half-grouted sleeve connections with insufficient grouting defect. *Constr. Build. Mater.* **2018**, *185*, 264–274. [CrossRef]
14. Sayadi, A.A.; Rahman, A.B.A.; Sayadi, A.; Bahmani, M.; Shahryari, L. Effective of elastic and inelastic zone on behavior of glass fiber reinforced polymer splice sleeve. *Constr. Build. Mater.* **2015**, *80*, 38–47. [CrossRef]
15. Li, Z.; Zheng, L.; Chen, C.; Long, Z.; Wang, Y. Ultrasonic detection method for grouted defects in grouted splice sleeve connector based on wavelet pack energy. *Sensors* **2019**, *19*, 1642. [CrossRef]
16. Li, D.; Liu, H. Detection of sleeve grouting connection defects in fabricated structural joints based on ultrasonic guided waves. *Smart Mater. Struct.* **2019**, *28*, 085033. [CrossRef]
17. Wu, C.; Yang, C.; Ma, S.; Xu, X. Feasibility study on grouting compactness detection in sleeves using piezoelectric transducers. *Appl. Sci.* **2019**, *10*, 149. [CrossRef]
18. Henin, E.; Morcous, G. Non-proprietary bar splice sleeve for precast concrete construction. *Eng. Struct.* **2015**, *83*, 154–162. [CrossRef]
19. Zhou, Z.; Wu, H.; Tan, C.; Xing, Y.; Gao, R. Comparison and a case study of test methods for sleeve grouting fullness in precast bridge piers. In *Life-Cycle Civil Engineering: Innovation, Theory and Practice*; CRC Press: Boca Raton, FL, USA, 2021; pp. 971–976.
20. Xu, B.; Fan, X.; Wang, H.; Zhou, S.; Wang, C.; Chen, H.; Ge, H. Experimental study on grout defects detection for grouted splice sleeve connectors using stress wave measurement. *Constr. Build. Mater.* **2021**, *274*, 121755. [CrossRef]
21. Liu, Y.-L.; Shi, J.-J.; Huang, J.-Q.; Wei, G.-S.; Wu, Z.-X. Grouting Defect Detection of Lapped Bar Connections Based on Impact-Echo Method. *Shock Vib.* **2019**, *2019*, 1934240. [CrossRef]
22. Liu, J.; Li, D.; Cui, X. Research status and future directions of defect detection in grouted splice sleeves: A review. *Constr. Build. Mater.* **2023**, *402*, 133010. [CrossRef]
23. Yao, F.; Ji, Y.; Tong, W.; Li, H.X.; Liu, G. Sensing technology based quality control and warning systems for sleeve grouting of prefabricated buildings. *Autom. Constr.* **2021**, *123*, 103537. [CrossRef]
24. Yao, F.; Abulikemu, A. Effect of impact source on detection quality in impact echo testing of sleeve grouting. *Mater. Test.* **2020**, *62*, 927–936. [CrossRef]
25. Yehia, S.; Abudayyeh, O.; Nabulsi, S.; Abdelqader, K. Detection of common defects in concrete bridge decks using nondestructive evaluation techniques. *J. Bridge Eng.* **2007**, *12*, 215–225. [CrossRef]
26. Du, Y.X.; Sun, X.L.; Yang, J.; Wu, Z.Q.; Bian, D.C. Research on the Sleeve Grouting Fullness Testing of Prefabricated Structures Based on Piezoelectric Method. *KSCE J. Civ. Eng.* **2024**, *28*, 1928–1946. [CrossRef]
27. T/CECS 683-2020; Technical Specification for Inspection of Sleeve Grouting Quality of Precast Concrete Structure. China Architecture & Building Press: Shanghai, China, 2020.
28. He, A.; Zheng, D.C.; Cai, J.; Chen, Q.J.; Tang, X.L.; Ye, J.B. Experimental investigation on the horizontal impact response of precast RC bridge piers with grouted sleeve connections considering defects. *Eng. Struct.* **2024**, *316*, 118617. [CrossRef]
29. Li, B.; Cao, J.; Xiao, J.Z.; Zhang, X.C.; Wang, H.F. Robotic Impact-echo Non-Destructive Evaluation based on FFT and SVM. In *Proceedings of the 11th World Congress on Intelligent Control and Automation*, Shenyang, China, 29 June–4 July 2014; pp. 2854–2859.
30. Yao, F.; Chen, G.Y.; Abula, A. Research on signal processing of segment-grout defect in tunnel based on impact-echo method. *Constr. Build. Mater.* **2018**, *187*, 280–289. [CrossRef]
31. Yeh, P.L.; Liu, P.L. Application of the wavelet transform and the enhanced Fourier spectrum in the impact echo test. *NDT E Int.* **2008**, *41*, 382–394. [CrossRef]
32. Tajari, M.; Shekarchi, M.; Sadri, A. Use of Impact-echo Technique for Detection of Distributed Damage in Concrete due to Alkali-silica Reactivity. *Mater. Eval.* **2011**, *69*, 881–890.

33. Yuan, L.B.; Jin, W.; Zhou, L.M.; Lau, K.T. The temperature characteristic of fiber-optic pre-embedded concrete bar sensor. *Sens. Actuators A-Phys.* **2001**, *93*, 206–213. [CrossRef]
34. Zhang, H.; Li, J.J.; Kang, F.; Zhang, J.A. Monitoring depth and width of cracks in underwater concrete structures using embedded smart aggregates. *Measurement* **2022**, *204*, 112078. [CrossRef]
35. Chaudhary, M.T.A. Effectiveness of Impact Echo testing in detecting flaws in prestressed concrete slabs. *Constr. Build. Mater.* **2013**, *47*, 753–759. [CrossRef]
36. Ohtsu, M.; Watanabe, T. Stack imaging of spectral amplitudes based on impact-echo for flaw detection. *NDT E Int.* **2002**, *35*, 189–196. [CrossRef]
37. Salazar, A.; Vergara, L.; Igual, J.; Gosalbez, J. Blind source separation for classification and detection of flaws in impact-echo testing. *Mech. Syst. Signal Process.* **2005**, *19*, 1312–1325. [CrossRef]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Risk Analysis of Service Slope Hazards for Highways in the Mountains Based on ISM-BN

Haojun Liu ¹, Xudong Zha ^{1,*} and Yang Yin ²

¹ School of Traffic and Transportation Engineering, Changsha University of Science and Technology, No. 2-960 Wanjiali Road (South), Yuhua District, Changsha 410114, China; liuhj607@163.com

² School of Civil Engineering, Central South University, 68 South Shaoshan Road, Changsha 410075, China; yinyang@csu.edu.cn

* Correspondence: zhaxd410@163.com

Abstract: To effectively mitigate service slope disaster risks in mountainous areas and enhance the overall safety of highway operations, based on the geological and structural characteristics of slopes, considering slope technical conditions, overall stability, and potential disaster consequences, 25 important influencing factors are systematically identified. The identification process integrates insights from the relevant literature, expert opinions, and historical disaster maintenance records of such slopes. An integrated approach combining Interpretive Structural Modeling (ISM) and Bayesian Networks (BNs) is utilized to conduct a quantitative analysis of the interrelationships and impact strength of factors influencing the disaster risk of mountainous service highway slopes. The aim is to reveal the causal mechanism of slope disaster risk and provide a scientific basis for risk assessment and prevention strategies. Firstly, the relationship matrix is constructed based on the relevant prior knowledge. Then, the reachability matrix is computed and partitioned into different levels to form a directed graph from which the Bayesian network structure is constructed. Subsequently, the expert's subjective judgment is further transformed into a set of prior and conditional probabilities embedded in the BN to perform causal inference to predict the probability of risk occurrence. Real-time diagnosis of disaster risk triggers operating slopes using backward reasoning, sensitivity analysis, and strength of influence analysis capabilities. As an example, the earth excavation slope in the mountainous area of Anhui Province is analyzed using the established model. The results showed that the constructed slope failure risk model for mountainous operating highways has good applicability, and the possibility of medium slope failure risk is high with a probability of 34%, where engineering geological conditions, micro-topographic landforms, and the lowest monthly average temperature are the main influencing factors of slope hazard risk for them. The study not only helps deepen the understanding of the evolutionary mechanisms of slope disaster risk but also provides theoretical support and practical guidance for the safe operation and disaster prevention of mountainous highways. The model offers clear risk information, serving as a scientific basis for managing service slope disaster risks. Consequently, it effectively reduces the likelihood of slope disasters and enhances the safety of highway operation.

Keywords: mountain highways; service slope disaster; risk evaluation; interpretive structural modeling; Bayesian network

1. Introduction

According to the statistics of the “Census of Highway and Waterway Bearing Bodies at Comprehensive Risk of Natural Disasters” completed by China in 2022, more than 220,000 pieces of highway disaster risk data have been collected nationwide, and more than 30,000 km of disaster sections have been cumulatively disposed of along highways during the 13th Five-Year Plan period [1]. Highways are linear projects with long spans and varying geological conditions along the route, thus forming many fills and cuts with varying conditions for subgrade slopes. The safety risk analysis of these slopes has always been one of the key issues in highway operation. In recent years, disasters such as collapses and landslides on highways have caused significant destruction to public and private property, traffic disruptions, and even casualties [2,3]. In July 2020, affected by heavy rainfall, the slope at the entrance of Zhuling tunnel on the Ningji Highway in the southern mountainous area of Anhui Province was washed away, blocking traffic for 3 h. In October 2021, heavy rains caused landslides and mudslides on many slopes of highways in Sichuan, disrupting highway traffic and resulting in significant economic losses [4]. In May 2024, a landslide occurred at the embankment slope of the Meizhou to Dapu Expressway, leading to the interruption of traffic in both directions, with a death toll as high as 48 people. This suggests that slope hazards along highways not only pose a significant hazard to access but also impede traffic, leading to excessive maintenance costs and even injuries. Obviously, it is of great significance to scientifically and effectively analyze the disaster risk of service slopes, establish corresponding risk evaluation models, objectively assess the current situation of service slopes, identify key influencing factors, and prevent and resolve major risks.

BN is a widely applied Artificial Intelligence (AI) technology, often referred to as belief networks, probabilistic networks, or causal networks, and has been extensively utilized in various fields, including dam risk analysis, environmental management, and construction management [5,6]. Accordingly, the study comprehensively considers the geological and structural characteristics of slopes, environmental features, and management factors along the mountainous operational highways. Drawing from historical disaster maintenance data for multiple service slopes of mountain highways in Anhui Province, China, and incorporating expert opinions, the fishbone analysis method was used to identify 25 influencing factors. It is challenging to construct a BN model with unknown structures or ambiguous parameters in the absence of complete historical data. ISM is introduced from a system perspective in order to fully analyze the relationship between factors and subdivide the complex system layer by layer through calculation to form a logically clear hierarchical relationship structure. Then, the ISM is converted into a BN model to calculate the overall risk level of service slope disasters. Subsequently, the key influencing factors and causal chain of slope hazard risk are determined through backward reasoning, sensitivity analysis, and strength of influence. The research results effectively address the complex interrelationships among various factors and provide systematic technical support for the risk assessment, early warning, and prevention of slope disasters on mountainous operational highways.

The remainder of the paper is organized as follows: Section 2 is the literature review. Section 3 carries out the analysis and selection of factors influencing the risk of slope hazards. Section 4 presents the risk analysis network construction and analysis method proposed. Section 5 takes the soil excavation slopes in service of a mountainous operating highway in Anhui, China, as the object, uses the method proposed to identify key risk influencing factors and relationships, and conducts backward reasoning, sensitivity analysis, and impact intensity analysis and validation. Section 6 is the conclusion. The proposed analysis framework is shown in Figure 1.

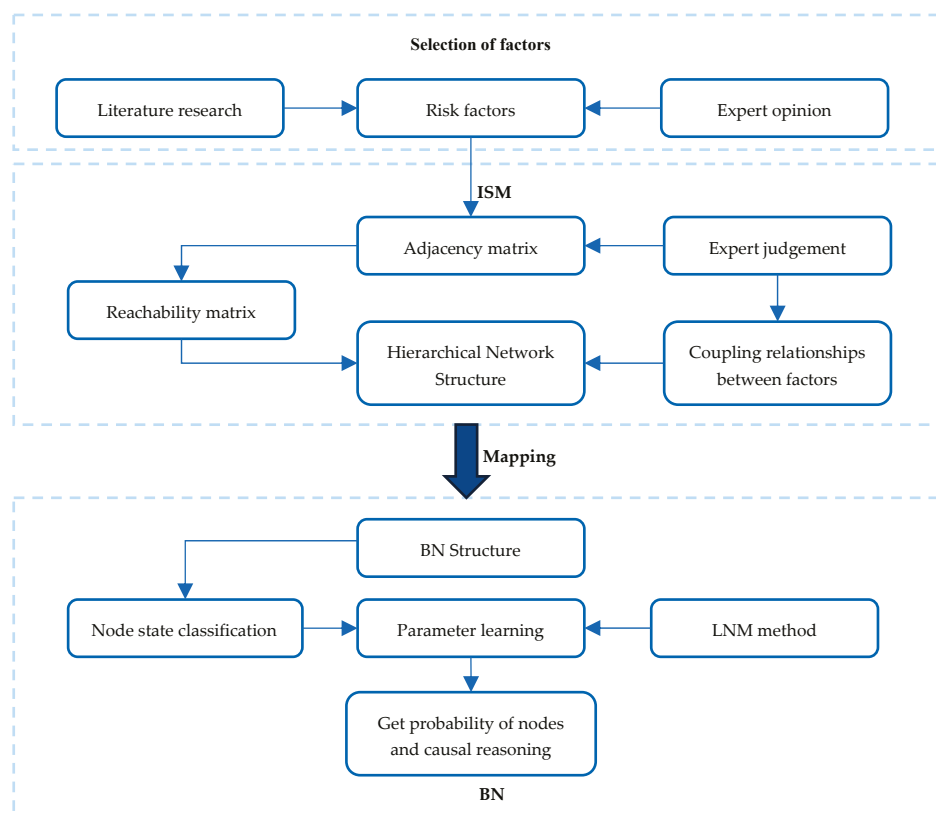


Figure 1. The proposed methodology flowchart.

2. Literature Review

Highway slope disaster (HSD) risk analysis is an effective method for disaster risk prevention and management, and accurate identification of potential systematic risk influencing factors is a prerequisite for establishing a reliable risk assessment model [7]. The causes of slope hazards are complicated. Geotechnical properties, slope gradient, slope height, heavy rainfall, and human disturbance are all important factors leading to slope disasters [8–11]. During heavy rainfall, the infiltration capacity of the slope is not sufficient to absorb all the rainwater into the soil. As a result, the rainwater that cannot infiltrate into the soil flows over the surface of the slope in the form of runoff, and it is obvious that both rainwater permeability and runoff undermine the stability of the slope [12,13]. Zhao et al. [14] found that, due to insufficient understanding of slope structures or inaccurate judgments of slope disaster, some slopes have poor protective effectiveness despite the implementation of protective measures. A study by Amarasinghe et al. [15] showed that rainfall is the main trigger for landslides and that about 90% of the landslide fatalities worldwide are due to rainfall-induced landslides. In previous studies related to highway slope risk, scholars at home and abroad have explored various methods to quantify slope disaster risks, such as the traditional slope stability evaluation model based on limit equilibrium [16] and finite element numerical simulation [17]. These methods have achieved notable success in slope mechanical behavior analysis and stability assessment. However, the level of slope disaster risk is affected by the joint influence of various uncertain factors inside and outside. It is insufficient to comprehensively capture the complexity and evolutionary characteristics of slope disaster risk using only deterministic mathematical modeling methods based on the principle of geotechnical characteristics design. Zhang et al. [18] proposed a risk assessment of the stability of expansive soil slopes based on Fuzzy Hierarchical Analysis (FAHP), but there are deficiencies in the risk evolution analysis of multi-factor coupling. Lin et al. [19] developed a quantitative risk assessment in terms of probabilistic loss for the cutting slope

risks during the design phase. This approach enabled the quantitative estimation and comparison of potential risk losses under different working conditions, but their study is largely confined to static risk analysis. Niu et al. [20] constructed a highway slope stability evaluation model integrating Principal Component Analysis (PCA), Genetic Algorithm (GA), and Support Vector Machine (SVM), providing a preliminary theoretical foundation for highway slope management. However, the method exhibits a strong dependency on data and fails to effectively reveal the intrinsic correlation among factors. Yin et al. [21] carried out spatial prediction of slope stability for mountain highways based on GCG-CNN with the support of the Geographic Information System (GIS), which provided a theoretical basis for improving the disaster resistance of highway and regional disaster prevention and mitigation capacity, but it focuses on spatial distribution characteristics and lacks sufficient analysis of the interactions among multiple factors.

BN consists of two parts: Directed Acyclic Graph (DAG) and conditional probability tables (CPTs), which can effectively characterize causal relationships among influencing factors through probabilistic reasoning [22]. Zhang et al. [23] developed a multi-index serviceability assessment model for dams based on BN, and the results showed that it has high reliability in assessing complex systems. Tang et al. showed that the BN model is an effective method for dam risk analysis. However, BN still has certain limitations in practical applications. Firstly, the construction of its network structure is highly dependent on domain knowledge, which may lead to subjective deviations. Secondly, the method faces challenges in directly addressing systems with numerous variables and intricate relationships, especially when the hierarchical structure among factors is ambiguous, thereby constraining its modeling efficiency and accuracy. ISM is a widely used structural modeling analysis method in systems engineering, particularly suitable for systematic analysis with numerous variables, complex relationships, and unclear hierarchy [24]. As a primarily qualitative analysis method, ISM systematically categorizes key influencing factors into distinct hierarchical levels, revealing their interrelationships and constructing a multi-level directed graph structure. This methodology effectively clarifies the interaction pathways among factors and simplifies complex relationships, thereby enhancing the overall structural clarity. This characteristic of ISM provides a clear structural framework for BN to make up for the limitations of BN modeling. Zhang et al. [25] developed a prefabricated building quality evaluation model by introducing the ISM-BN method, demonstrating the applicability of it. Shen et al. [26] combined ISM with BN to realize early warning of schedule risk in international railway projects and demonstrated the potential of combining these two methods.

Mountainous highway service slope systems are characterized by large scale, complex environments, and dynamic nature. Slope instability disasters typically result from the combined effects of multiple risk factors with intricate interrelationships. Traditional single methods often fail to comprehensively characterize the risk evolution process. The ISM-BN integrated analytical method addresses these limitations through ISM's hierarchical analysis capability, which effectively clarifies complex interrelationships between influencing factors and establishes a clear structural framework. Meanwhile, BN's probabilistic reasoning capability further enhances dynamic analysis and certainty assessment of the risk evolution process.

3. Selection of Factors for Operating HSD Risk in Mountainous Areas

Slope disasters in mountainous operational highways result from the synergistic effects of multiple risk factors. The formation mechanism is complex and dynamically evolved, involving multidimensional interactions among environmental, geological, engineering, and managerial factors. Therefore, the selection of factors plays a crucial role in the entire

risk analysis and risk management system for operating HSD risk in mountainous areas, serving not only as the theoretical foundation for risk assessment but also as the scientific basis for risk prevention and control practices.

3.1. Risk of HSD in Mountainous Areas

Highway slopes consist of the slope itself, water and retaining protection, and drainage structures, all working together to achieve their specific functions. The slope disaster risk of mountainous highways in operation refers to the damage or destabilization disasters that may occur during the operation of slopes along mountainous highways, which may lead to adverse consequences such as reduced durability of the project, obstruction of highway traffic, property damage, and even casualties. According to the Technical Specification for Maintenance of Highway Subgrade (JTG 5150-2020) [27], combined with the research results of Xu et al. [28], the service slope hazards are mainly related to slope scouring, fragmentation and collapse, damage to supporting facilities, partial collapse and landslides, etc.

3.2. Influencing Factors of HSD Risk in Mountainous Areas

It was found that the main influencing factors of disasters on service slope include meteorological environment, slope geotechnical characteristics, slope height, slope support structures, slope drainage facilities, design quality, construction quality, maintenance quality, and travel load by analyzing the historical disaster events of multiple typical service slopes of several operating highways in the Dabie mountain area and the southern Anhui mountain area of Anhui from 2015 to 2020 [28]. The research by Lan et al. [29] showed that the slope hazard risk is sensitive to engineering geological conditions and topographic and geomorphic conditions. The results obtained by [30] have indicated that most of the highway retaining wall failures around the world occurred due to missing or inadequate drainage systems. In addition, other scholars have also identified numerous risk factors that affect the performance of highway slopes in their studies on specific slopes, as shown in Table 1.

Table 1. Research on the influencing factors of expressway slope risk.

Literature	Relevant Influencing Factors
[31,32]	Rainfall intensity, slope gradient
[13]	Surface runoff, groundwater
[33]	Engineering geological conditions, extreme climate events, human activities
[34]	Slope disease history, damage to supporting structures, damage to drainage facilities, geological engineering conditions, hydrogeological conditions
[35]	Micro-topography, hydrological conditions, average annual precipitation days, geotechnical type, vegetation coverage
[36]	Engineering geological conditions, topography, construction quality, slope deformation characteristics

It is clear that the formation of slope hazards may be affected by a combination of one or more of these factors. In order to make the relevant influencing elements comprehensive and representative, based on the existing literature, after several rounds of expert discussion, the fishbone diagram analysis method was chosen to sort out the influencing factors of slope disaster risk for mountain highways. The influencing factors of slope disaster risks of the mountainous highway in service can be categorized into four aspects, namely environmental factors, slope technical conditions, disaster-bearing sets, and management factors. The fishbone diagram was constructed to analyze the disaster risk of the service highways in mountains, as shown in Figure 2.

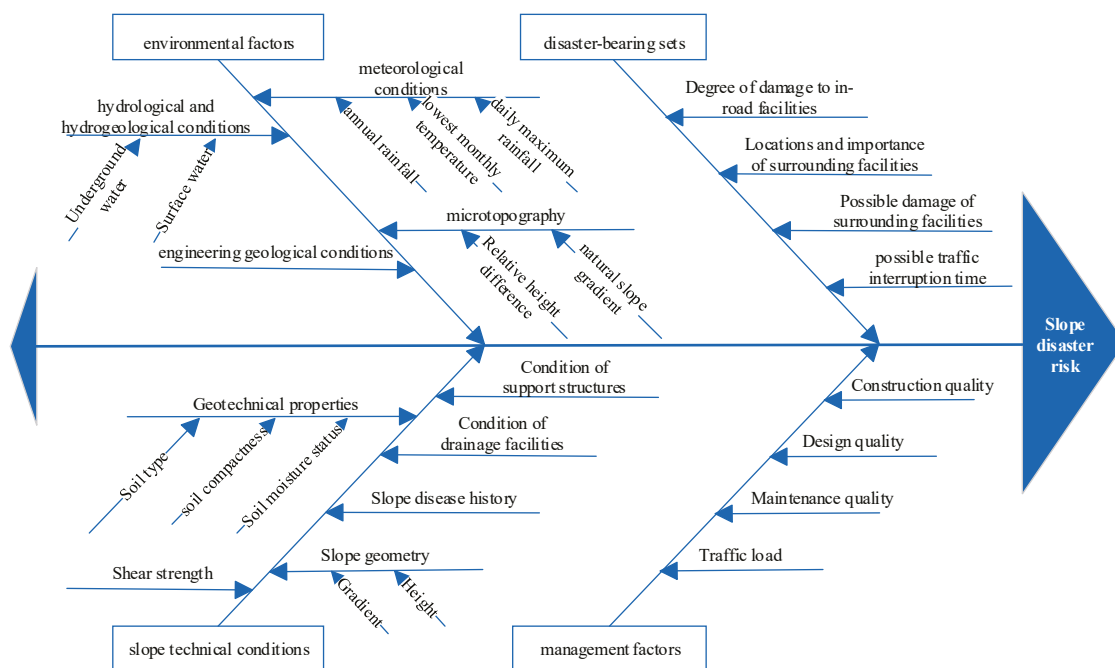


Figure 2. Fishbone diagram for disaster risk analysis of service soil cut slopes.

- (1) Environmental factors specifically include the influences of six factors: engineering geological conditions X_1 , hydrological and hydrogeological conditions X_2 , microtopography and landform X_3 , annual precipitation X_4 , daily maximum rainfall X_5 , and the lowest monthly average temperature X_6 .
- (2) The technical conditions of the slope specifically include the influences of 10 factors: soil properties X_7 , soil compaction X_8 , soil moisture status X_9 , shear strength X_{10} , slope height X_{11} , slope gradient X_{12} , slope disease history X_{13} , support structure status X_{14} , drainage facility status X_{15} , and slope stability level X_{16} .
- (3) Management factors specifically include the influences of four factors: design quality X_{17} , construction quality X_{18} , maintenance quality X_{19} , and traffic load rating X_{20} .
- (4) Disaster-bearing set. It is of no practical significance to discuss risk without considering the disaster-bearing objects. The paper mainly takes into account the influences of five factors: the possible damage degree of in-road facilities X_{21} , the location of surrounding facilities X_{22} , the importance degree of surrounding facilities X_{23} , the possible damage degree of surrounding facilities X_{24} , and the possible traffic interruption time X_{25} , which reflect the degree of harm caused by the consequences of the slope instability event.

4. Construction of ISM-BN Model

Based on this, this paper combines ISM with BN to scientifically and effectively establish a risk assessment model. According to the characteristics of the influencing factors of disaster risk in the actual situation, it accurately identifies the key risk factors and provides systematic technical support for risk management and control.

4.1. Constructing ISM for HSD Risk Factors

There are numerous influencing factors for the slope disaster risk of mountainous operation highways, and the relationship between the factors is complex. Taking the service soil excavation slope disaster risk as an example, the ISM model is used to sort out the relationship between the factors. The specific steps are as follows.

Step 1: Establish the adjacency matrix **A**

Based on the risk factors extracted in Section 3.2, obtain the relationships among the factors according to the expert judgment, an adjacent matrix $A = (a_{ij})_{n \times n}$ is established. Among them, $a_{ij} = 1$ indicates that factor X_i influences factor X_j , and vice versa, $a_{ij} = 0$.

Step 2: Calculate the reachability matrix $\mathbf{R}$

Import the adjacent matrix $\mathbf{A}$ into the MATLAB 2022 software and calculate the reachable matrix $\mathbf{R}$ according to the Boolean operations rules.

$$\mathbf{R} = (\mathbf{A} + \mathbf{I})^{n-1} \quad (1)$$

where $\mathbf{I}$ is the unit matrix, n is the number of variables.

Step 3: Divide the reachability matrix $\mathbf{R}$ into different levels

Firstly, the reachability set $M(r_i)$, the antecedent set $P(r_i)$ as well as the common set $C(r_i)$ of the reachability matrix $\mathbf{R} = (r_{ij})_{n \times n}$ should be developed. $M(r_i)$ can be expressed as $M(r_i) = \{r_j | r_j \in R, r_{ij} = 1\}$, that composed of indicator i itself and other indicators in the row, the $r_{ij} = 1$. $P(r_i)$ can be expressed as $P(r_i) = \{r_j | r_j \in R, r_{ji} = 1\}$, that composed of indicator i itself and other indicators in the column, the $r_{ji} = 1$. The definition formula of $C(r_i)$ is $C(r_i) = \{r_j | r_j \in R, r_{ij} = 1, r_{ji} = 1\}$, where $i, j = 1, 2, \dots, n$, and it represents the common part of R_i in the reachable and antecedent sets, i.e., the intersection.

Then, indicators at level i can be obtained, and these indicators should be removed from the pool of indicators. Repeat the above steps until the level division of all indicators is completed. When the iteration ends, the hierarchical division of factors affecting the disaster risk of soil cut slopes will be obtained.

Step 4: Plot the multi-level recursive directed graph

Based on the different levels of the reachability matrix $\mathbf{R}$, nodes and arcs are used to construct a directed graph of the influencing factors. The aim is to describe the interdependence of the indicators in the horizontal direction and the mutual influence between two adjacent levels of them in the vertical direction. The directed hierarchical interpretive structure shows the paths through which the risk factors act and the action mechanism of the step-by-step transmission of the risk factors, as shown in Figure 3.

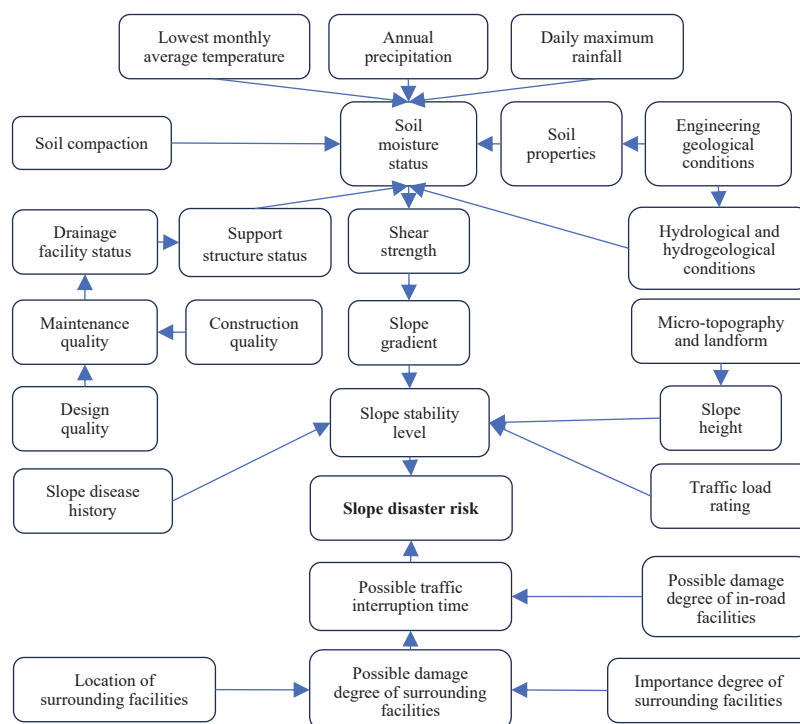


Figure 3. The directed graph based on ISM.

4.2. Constructing the BN Model of HSD Risk

Construct the BN according to the following steps.

Step 5: Form the Bayesian network

Based on the arcs between the risk factors of the disaster system for service soil cut slopes of expressways in mountains obtained by ISM, the ISM hierarchical structure is transformed into a corresponding Bayesian network topology as the basis of the BN inference model. From Figure 2, it can be seen that the ISM forms a directed graph without redundant and circular arc links, which are not allowed in BN [37]. The nodes in the directed graph are transformed into the nodes of the Bayesian network topology.

Step 6: Determine network parameters

(1) Node status classification

Firstly, the relevant variables are handled through fuzzy logic. According to the Specification for Risk Assessment of Geological Hazard (GB/T 40112) [38] and Census technical handbook for highway hazard-affected bodies in natural disaster comprehensive risk, with reference to the research of Xu et al. [28] and Liu et al. [39,40], and combined with the opinions of experts, the node states are divided into at most five states, which are uniformly represented by “R1”, “R2”, “R3”, “R4”, and “R5”, comprehensively considering the actual situation of factors and the parameter learning cost. The states are ranked from strong to weak depending on the intensity of their influence. The specific classification is shown in Table 2. These levels correspond to the states of the nodes in the BN topology to better reflect the level of slope disaster risk.

Table 2. Rules for the division of Bayesian network node status.

Node	R1	R2	R3	R4	R5
Engineering geological conditions X_1	Poor	Moderate	Simple	—	—
Hydrological and hydrogeological conditions X_2	Poor	Moderate	Simple	—	—
Micro-topography and landform X_3	Complicated	Moderate	Simple	—	—
Annual precipitation X_4 (mm)	≥ 1400	$1000 \leq X_4 < 1400$	$600 \leq X_4 < 1000$	$X_4 \leq 600$	—
Daily maximum rainfall X_5 (mm)	≥ 250	$100 \leq X_5 < 249.9$	$50 \leq X_5 < 99.9$	$25 \leq X_5 < 49.9$	$X_5 \leq 24.9$
Lowest monthly average temperature X_6 (°C)	< 2	$2 \leq X_5 < 6$	$X_5 \geq 6$	—	—
Soil properties X_7	Clayey soil	Silt	Sand	Gravelly soil	—
Soil compaction X_8	Loose	Slightly dense	Moderately dense	dense	—
Soil moisture status X_9	Very wet	Wet	Slightly wet	—	—
Shear strength X_{10} ($\varphi' / ^\circ$)	$\varphi' \leq 25$	$25 < \varphi' \leq 40$	$\varphi' > 40$	—	—
Slope height X_{11} (m)	$X_{11} \geq 20$	$6 \leq X_{11} < 20$	$X_{11} < 6$	—	—
Slope gradient X_{12}	$X_{12} > 1:0.9$	$1:1.2 \leq X_{11} < 1:0.9$	$1:1.5 \leq X_{12} < 1:1.2$	$X_{12} < 1:1.5$	—
Slope disease history X_{13}	Severe	Moderate	Slight	None	—
Support structure status X_{14}	Structural instability	Local damage	Apparent damage	None	—
Drainage facility status X_{15}	Not connected to the external drainage system	Facility damage	Clogging	None	—

Table 2. Cont.

Node	R1	R2	R3	R4	R5
Slope stability X_{16}	Poor	Average	Good	—	—
Design quality X_{17}	Unqualified	Qualified	High-quality	—	—
Construction quality X_{18}	Unqualified	Qualified	High-quality	—	—
Maintenance quality X_{19}	Unqualified	Qualified	High-quality	—	—
Traffic load rating X_{20}	Extra-heavy and extremely heavy traffic	Heavy traffic	Light and moderate traffic	—	—
Possible damage degree of in-road facilities X_{21}	Severe	Moderate	Slight	None	—
Location of surrounding facilities X_{22}	Horizontal distance outside the top of the slope ≤ 0.5 times the slope height	Horizontal distance outside the top of the slope (0.5, 1.0] times the slope height	Horizontal distance outside the top of the slope (1.0, 1.5] times the slope height	Horizontal distance outside the top of the slope > 1.5 times the slope height	—
Importance degree of surrounding facilities X_{23}	Security level 1st	Security level 2nd	Security level 3rd	—	—
Possible damage degree of surrounding facilities X_{24}	Severe	Relatively severe	Moderate	Slight or none	—
Possible traffic interruption time X_{25} (T/h)	$T \geq 24$	$4 \leq T < 24$	$1 \leq T < 4$	$T < 1$	—
Slope disaster risk T	Extremely high	High	Moderate	Low	—

(2) Parameter learning

Parameter learning of the Bayesian network (BN) is the process of determining the conditional probability table (CPT) of the network. Due to the lack of systematic statistical data in the field of highway service slopes in mountains, it is difficult to conduct ideal parameter learning for BN using machine learning. The paper established a BN model by means of a parametric learning method based on expert knowledge and historical experience. The BN model mainly relies on the prior probability values of root nodes. Considering that experts usually tend to give a fuzzy interval rather than an exact value when estimating the probability of an event occurring. Moreover, factors such as experts' knowledge reserves and subjective concepts can also lead to a certain degree of uncertainty in the given results. Therefore, it is determined by applying the IPCC ranking criteria [41] (as shown in Table 3) combined with the decision of the expert group. The prior probabilities of root nodes are characterized by inviting several experts to give fuzzy intervals. Then, drawing on the research results about expert weights division [42] (as shown in Table 4), the evaluation results are integrated and normalized to obtain the priori probability of the root nodes in the network.

Table 3. IPCC-tiered probability.

Grade	Probability Description	Probability Range (%)
1	Exceptionally unlikely	<1
2	Very unlikely	1~10
3	Unlikely	10~33
4	About as likely as not	33~66
5	Likely	66~90
6	Extremely likely	90~99
7	Virtually certain	>99

Table 4. Expert weight division.

Grade	Probability Range (%)	Weight
Level 1	Professor, researcher, and senior engineer: more than 30 years of service.	1
Level 2	Associate professor, associate researcher, senior engineer: 20~30 years of service.	0.9
Level 3	Associate professor, associate researcher, senior engineer: 10~20 years of service.	0.8
Level 4	Engineer, lecturer, assistant researcher: 5 to 10 years of service.	0.7
Level 5	Engineer, lecturer, assistant researcher: 1 to 5 years of service.	0.6

When there exists a “multiple causes leading to one effect” situation in the BN, the number of CPT parameters for the child nodes grows exponentially, which poses a greater challenge to parameter learning and diagnostic inference. Moreover, although the constructed BN model incorporates numerous relevant factors by integrating expert opinions, it may still be incomplete. To improve the validity and completeness of the method, the Leaky Noisy-Max (LNM) is introduced, which simplifies the construction of polymorphic CPT and also takes into account other factors that may be ignored or randomized in the model [43]. The calculation of conditional probability under the LNM method is as follows:

$$P_{leaky} = P(Y_L = y) = P(y|\bar{x}_1, \dots, \bar{x}_n) \quad (2)$$

$$P(Y \leq y|X) = \prod_i \left[\sum_{Y \leq y} P(Y = y|X = x_i) \right] \sum_{f_{\max}(Y, Y_L) \leq y} P(Y_L) \quad (3)$$

$$P(Y|X) = \begin{cases} P(Y \leq y|X) - P(Y \leq y-1|X) & , y \neq y_{\min} \\ P(Y \leq y|X) & , y = y_{\min} \end{cases} \quad (4)$$

where P_{leaky} presents the leaky probability, Y represents the child node variable, and y is the value taken by node Y . X represents the parent node variable, and x_i is the value taken by node X . $\bar{x}_i$ is the value not included in node X .

Step 7: Causal reasoning for the BN

The acquired parameters were imported into GeNIe 4.1 software, developed by the Decision Systems Laboratory at the University of Pittsburgh, which supports a variety of Bayesian network inference algorithms and displays structural changes through a straightforward visual interface. Through its causal inference function to calculate the distribution results of the posterior probability of the risk of non-root nodes, and with reference to the risk classification standard based on the ALARP principle [44], to achieve the classification evaluation of the service slope disaster risk for mountainous expressway is achieved. Secondly, key risk factors and risk transmission links are identified through risk diagnosis, a process that includes Setting the risk status of leaf nodes, obtaining the posterior probability of each non-leaf node under different risk levels through backward reasoning, and then screening out the risk factors that have a greater impact on the risk events. Setting the leaf nodes as the target nodes and identifying the risk factors that are more sensitive to the occurrence of risk events through the sensitivity analysis. Choosing “Euclidean” as the distance function and determining the interaction intensity among various risk factors with the help of the influence intensity analysis so as to facilitate the adoption of targeted risk prevention and control measures to weaken the scope and amount of the impact of key risk factors and risk transmission links in the network.

5. Results and Discussion

Based on the overall technical condition of the soil cut slopes of the operational highway in the Dabie and Southern Anhui Mountain areas of Anhui Province, a risk assessment team was formed by five experts from the field of highway engineering. They were sequentially numbered as Expert 1 to Expert 5, and their specific information is

detailed in Table 5. The prior probability values of each root node were determined, as shown in Table 6. Then, with reference to Equations (2)–(4), the conditional probability tables under the LNM model were determined. For example, the conditional probability table (CPT) of node X_{25} and its parent nodes X_{21} and X_{24} is shown in Table 7.

Table 5. Expert information.

Expert ID	Background	Years of Service	Weight
Expert 1	Professor	31	1
Expert 2	Senior engineer	19	0.8
Expert 3	Senior engineer	11	0.8
Expert 4	Lecturer	8	0.7
Expert 5	Engineer	5	0.6

Table 6. Prior probability values of root node in BN.

Node ID	Status	Evaluation Grade					Prior Probability	Node ID	Status	Evaluation Grade					Prior Probability
		Expert 1	Expert 2	Expert 3	Expert 4	Expert 5				Expert 1	Expert 2	Expert 3	Expert 4	Expert 5	
X_1	R1	3	3	4	4	4	0.2820	X_{13}	R2	4	4	3	4	3	0.3007
	R2	4	4	4	5	5	0.4553		R3	5	5	6	5	6	0.5862
	R3	3	3	4	4	4	0.2627		R4	2	2	3	3	3	0.0740
X_3	R1	5	4	4	4	5	0.6428	X_{17}	R1	3	3	2	2	2	0.0218
	R2	4	4	4	4	4	0.3372		R2	6	6	6	5	5	0.7025
	R3	3	3	2	2	3	0.0200		R3	4	4	3	3	3	0.2757
X_4	R1	3	3	3	2	2	0.0679	X_{18}	R1	3	3	3	2	2	0.0230
	R2	4	4	4	4	4	0.3060		R2	6	6	5	5	5	0.7238
	R3	5	5	6	6	6	0.5478		R3	4	4	3	3	3	0.2533
	R4	2	2	2	3	3	0.0783	X_{20}	R1	3	3	3	4	4	0.1985
X_5	R1	2	2	2	3	3	0.0687		R2	6	6	5	4	5	0.6173
	R2	4	3	3	3	4	0.2456		R3	4	3	3	2	2	0.1985
	R3	5	6	6	5	5	0.5494	X_{21}	R1	3	2	2	3	3	0.0906
	R4	2	2	3	3	3	0.0675		R2	4	4	3	3	3	0.2128
	R5	2	2	2	3	3	0.0687		R3	4	4	4	4	4	0.3046
X_6	R1	5	6	6	6	5	0.6279	X_{22}	R4	4	4	5	5	5	0.3920
	R2	3	4	3	4	4	0.2027		R1	2	2	2	3	1	0.0490
	R3	3	3	3	4	4	0.1694		R2	3	3	3	2	2	0.1148
X_8	R1	2	2	2	1	3	0.0364		R3	4	4	4	3	3	0.2817
	R2	4	4	3	3	3	0.2178		R4	6	6	5	6	5	0.5546
	R3	5	6	6	5	5	0.4904	X_{23}	R1	2	2	2	3	3	0.0703
	R4	4	4	4	3	3	0.2554		R2	3	3	4	4	3	0.2238
X_{13}	R1	2	2	3	3	3	0.0391		R3	6	6	6	5	5	0.6933

Table 7. CPT of the dependence relationship between node X_{25} and its parent nodes X_{21} and X_{24} .

Child Node	Parent Node		X_{21}			X_{24}			LEAK
	Status		R1	R2	R3	R1	R2	R3	
X_{25}	R1		0.3978	0.2736	0.2644	0.4960	0.2020	0.0959	0.0001
	R2		0.3567	0.2551	0.2326	0.3779	0.4565	0.1868	0.0004
	R3		0.1848	0.2551	0.2584	0.1088	0.2665	0.4316	0.001
	R4		0.0607	0.2162	0.2446	0.0173	0.0750	0.2857	0.9985

The prior probabilities of root nodes, along with the conditional probability tables, were input into the GeNIe 4.1 software. Subsequently, the distribution results of the posterior probabilities of non-root node risks were obtained through the BN causal inference function, as shown in Figure 4. The probability of the leaf node T “slope disaster risk” being extremely high is 13%, high is 28%, medium is 34%, and low is 25%. This indicates that under the combined effects of root nodes such as “annual precipitation”, “daily maximum rainfall”, “micro-topography and geomorphology”, “engineering geological conditions”, “traffic load rating”, “slope disease history”, “construction quality”, “importance degree of surrounding facilities”, etc., there is a relatively higher likelihood that the slopes of operational expressways in mountainous areas are at a medium risk level. In engineering

practice, the slopes of operational expressways in mountainous areas are so numerous that it is infeasible to conduct special designs on a case-by-case basis. Instead, the majority of them are constructed in accordance with engineering experience. Consequently, both the overall design quality and construction quality generally remain at a merely qualified level and fail to attain high quality. In addition, the probability of the slope maintaining a generally stable condition is 60%. This aligns with the widely recognized fact that as the service time of the slopes along operational expressways in mountainous regions extends, the reinforcement corrosion and concrete deterioration within the slope support and drainage structures take place. This, in turn, leads to the overall performance of the slope entering a state of gradual decline. The model graphically describes the quantitative mapping relationship between various influencing factors, presenting a clear and vivid picture of how they interact and relate to one another.

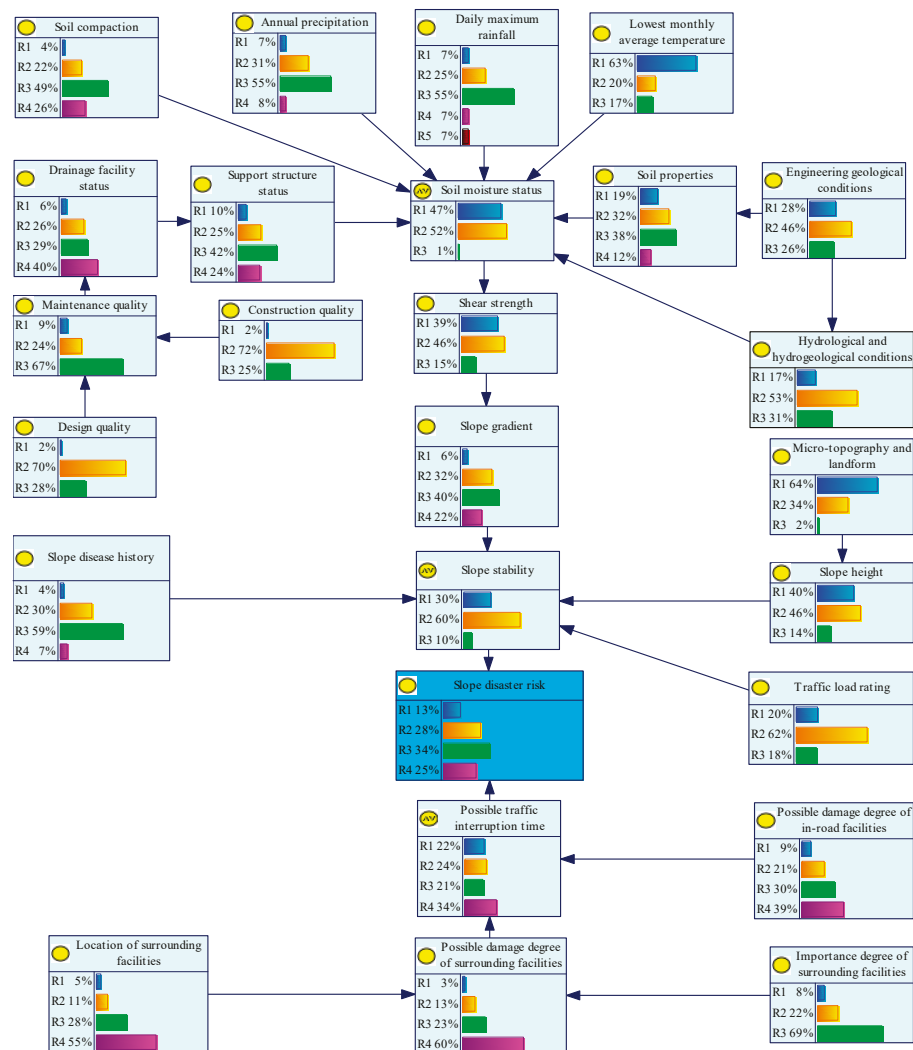


Figure 4. Bayesian network (BN) for service slope disaster risk.

To further diagnose the risk incentives, the reverse reasoning, sensitivity analysis, and influence intensity analysis functions of the Bayesian network were utilized to quantitatively study the slope disaster risk factors and risk transmission links and calculated the posterior probabilities, sensitivities, and intensities of risk transmission links of non-leaf nodes under the “very high” risk level, and the results are shown in Figure 5.

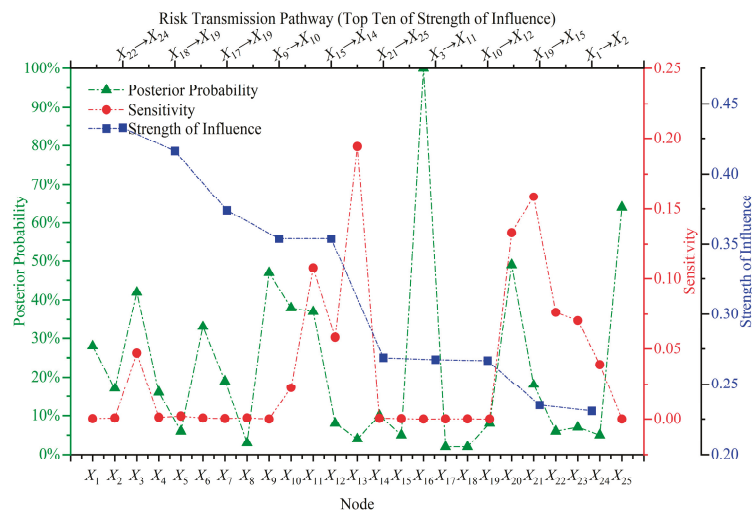


Figure 5. Results of BN backward inference, sensitivity analysis, and optimal causal chain analysis.

Root node factors are the focus of the risk diagnostic process. They have the attributes of risk sources and can be pre-empted with targeted risk prevention and control measures. As can be seen from Figure 4, the posterior probability of root nodes X_1 , X_3 , and X_6 was larger, respectively, at 28%, 66%, and 63%, which were risk factors leading to the risk of service slope hazards at an extremely high level of probability of a larger risk factor, significantly higher than the risk level of other root nodes, which requires special attention in risk management and control. The root nodes X_4 , X_5 , and X_{21} exhibit the highest sensitivity indices, specifically 0.1015, 0.2935, and 0.1553 in sequence. This indicates that a slight adjustment to the grading levels of these sensitivity factors would have a significant impact on the magnitude of in-service slope disasters. The influence intensities of $X_{22} \rightarrow X_{24}$, $X_{18} \rightarrow X_{19}$, and $X_{17} \rightarrow X_{19}$ were conspicuously greater than those of other risk transmission links. These particular links exert a more significant impact on the overall risk network. Consequently, the risk level can be effectively diminished by controlling the risk factors situated at both end termini of these two links, thereby severing these links.

Based on the above analyses, it is found that among the risk influencing factors of root nodes, the engineering geological conditions (X_1), micro-topography and landform (X_3), and the lowest monthly average temperature (X_6) had the greatest impact on disasters of service soil cut slope. These particular elements thus stand out as the key risk factors that lead to slope disaster events. Annual precipitation (X_4), daily maximum rainfall (X_5), and the possible damage degree of in-road facilities (X_{21}) held a relatively significant potential influence over the risk of slope disasters. The above risk diagnosis results were consistent with the theoretical reality of highway slope engineering, verifying the feasibility and effectiveness of the model.

To further verify the feasibility and effectiveness of the model, the information variables related to past real disaster events of service soil cut slopes on mountainous area expressways in Anhui Province were introduced into the BN model for scenario analysis. Taking the cut slope K466 + 000 on the downlink of the Tongling to Chizhou Expressway section as an example, the specific information of event factor variables is shown in Table 8.

The highway was completed and opened to traffic at the end of 2006, with excellent construction quality and good operation. The project is located in the transition zone from the Yangtze River floodplain to low hills. It has a two-level slope. The first-level slope was protected by a mortar rubble facing wall with a slope height of 5.5 m, and the second-level slope was protected by grass planting, with a slope height of 10 m. The subgrade side ditch was a blind drain, the slope covered with silty clay, which is expansive, and the vegetation

on the slope grows ordinarily. There is a high-voltage power tower outside the top of the slope. In July 2016, eroded by the extraordinary rainstorm, the top of the slope slid down. The landslide buried the emergency lane on the roadway, and the traffic was not interrupted for the time being. This section was identified as a high-risk slope. The relevant factors of slope disaster risk were subjected to grade normalization processing and then imported into the Bayesian network model for quantitative analysis. The running results are shown in Figure 6. Obviously, the probability that the slope has a high hazard risk is higher at 33%, which is consistent with the results of the risk at the project site, and the reasoning process is clear and straightforward.

Table 8. Value-taking of variable state for the landslide event of service soil excavation slope K466 + 000.

Node	Detailed Information	Node	Detailed Information	Node	Detailed Information
X ₁	—	X ₂	—	X ₃	—
X ₄	1871.1 mm	X ₅	131.6 mm	X ₆	4.1 °C
X ₇	—	X ₈	—	X ₉	—
X ₁₀	—	X ₁₁	15.5 m	X ₁₂	—
X ₁₃	Slight	X ₁₄	Local damage	X ₁₅	—
X ₁₆	—	X ₁₇	—	X ₁₈	—
X ₁₉	—	X ₂₀	Extra-heavy and extremely heavy traffic	X ₂₁	Slight
X ₂₂	Horizontal distance outside the top of the slope (0.5, 1.0] times the slope height	X ₂₃	Security level 1st	X ₂₄	—
X ₂₅	—	T	—	—	—

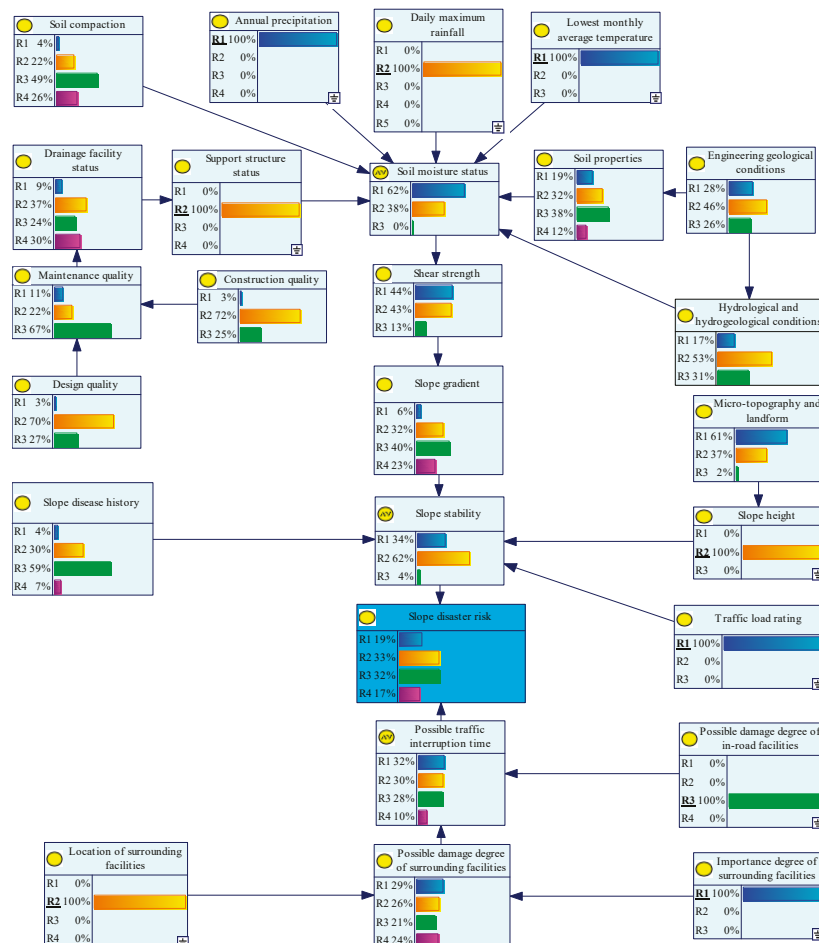


Figure 6. BN reasoning for disaster risk on service soil cut slope K466 + 000.

6. Conclusions

Highway slopes are inevitably subject to varying degrees of disaster risk while exerting great engineering benefits during the long-term operation. These risks may pose a serious threat to traffic safety and infrastructure stability. To effectively mitigate the occurrence of slope disaster accidents on operational expressways in mountains, the paper proposes a quantitative analysis method based on ISM and BN. The proposed method systematically analyzes the disaster risk of soil excavated slopes, identifying critical influencing factors and their correlations. This methodology provides actionable insights for risk control and disaster prevention.

- (1) Based on the historical disaster information of service slopes and the relevant research literature, a total of 25 factors affecting the disaster risk were identified from four aspects, namely, environmental factors, slope technical conditions, disaster-bearing sets, and management factors by means of fishbone analysis. A risk impact factors list was established, thereby enriching the methodological framework for risk factor identification for slope disasters in mountainous operational highways. This advancement fills critical gaps in current research regarding comprehensive risk factor identification, establishing a substantial theoretical and practical basis for future studies on slope disaster risk assessment and mitigation strategies in mountainous highway environments.
- (2) Apply the ISM-BN method to construct a slope disaster risk assessment model for mountainous highways. Through the multi-level directed interpretation structural model of ISM, the hierarchical relationships and interaction pathways among influencing factors are revealed. Leveraging BN's probabilistic reasoning capabilities, it quantitatively assesses node state probabilities, importance measures, sensitivity indices, and risk transmission pathways strength, enabling dynamic analysis and quantitative evaluation of risk evolution processes. Furthermore, the research introduces a novel fuzzy number-based method for calculating root node prior probabilities and incorporates the Leaky Noisy-Max model for conditional probability table determination, significantly enhancing model applicability and accuracy. Compared with conventional analytical approaches, the integrated ISM-BN methodology provides a more comprehensive framework for characterizing multi-factor interactions and their impacts on risk evolution within complex systems, establishing a new paradigm for slope disaster risk assessment.
- (3) Taking the soil-cutting slopes of operational highways in mountainous areas of Anhui Province as an example, the simulation analysis and empirical validation of the constructed risk assessment model were conducted. It was found that engineering geological conditions X_1 , micro-topography and landform X_3 , and minimum monthly average temperature X_6 were the key risk factors affecting in-service earth excavation slope hazards, and annual precipitation X_4 , daily maximum rainfall X_5 , and possible damage to in-road facilities X_{21} had a greater potential impact on it, which can be targeted for prevention and control in daily maintenance. These findings provide targeted guidance for routine maintenance and risk management of expressway slopes in mountainous areas. Further analysis revealed a strong correlation between the model's outputs and field-based risk assessment results, confirming the model's applicability and reliability.

This research not only deepens the understanding of slope disaster risk evolution mechanisms but also provides theoretical support and practical guidance for safety operations and disaster prevention in mountainous expressways. The integrated ISM-BN analytical method enables the effective identification of high-risk factors and optimization of risk management strategies, thereby reducing the probability of slope disasters and

enhancing the operational safety of highway systems. Nevertheless, there are several limitations that warrant consideration. Primarily, the model's construction relies heavily on historical data and expert knowledge, potentially introducing subjective biases that may compromise its objectivity and generalizability. Secondly, constrained by data accessibility, the model's generalizability across different scenarios remains insufficiently validated. Furthermore, the model exhibits limitations in its capacity for real-time updating of dynamic risk factors, consequently restricting its ability to accurately characterize the temporal evolution of slope disaster risks. To enhance the model's robustness and practical applicability, future research will focus on the following aspects:

- (1) With the development of modern highway systems and the accumulation of certain slope-related data, we will optimize the structural and parametric learning processes of the model based on the updated accident database, combined with machine learning and deep learning techniques, to reduce the dependence on expert knowledge in order to enhance the robustness and practical applicability of the model;
- (2) The proposed methodology could be enhanced through the integration of multi-source heterogeneous data, enabling the development of a dynamic risk assessment model based on data fusion techniques. This approach would notably improve the model's adaptability to complex environmental variations, facilitating real-time monitoring and early warning capabilities for slope disaster risks;
- (3) While the paper focuses on slope disaster risk management on operational expressways in mountains, future research could expand its scope to encompass the entire life-cycle, including investigation, design, and construction stages. This expansion would facilitate the development of a comprehensive risk management system covering the complete "investigation–design–construction–operation" process. By integrating and sharing data from multiple phases, a holistic risk assessment model could be established to elucidate risk factor transmission and evolution mechanisms across different stages. Such advancement would optimize risk prevention strategies and significantly improve the overall effectiveness of slope disaster risk management in mountainous expressway systems.

Author Contributions: Conceptualization, H.L. and X.Z.; methodology, H.L. and Y.Y.; software, H.L.; validation, H.L. and Y.Y.; formal analysis, X.Z.; investigation, H.L.; resources, X.Z.; data curation, H.L.; writing—original draft preparation, H.L.; writing—review and editing, X.Z. and Y.Y.; visualization, H.L.; supervision, X.Z. and Y.Y.; project administration, X.Z. All authors have read and agreed to the published version of the manuscript.

Funding: This work was supported by the National Natural Science Funds of China [grant number 51878077]; the Science and Technology Progress Project of the Department of Transport in Anhui Province, China [grant number 201839]; and the Postgraduate Scientific Research Innovation Project of Hunan Province, China [grant number CX20200824].

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed at the corresponding author.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Ministry of Transport of the People's Republic of China. Reduce Disaster Risks and Ensure the Safety of Highway Travel. 2022. Available online: https://www.mot.gov.cn/jiaotongyaowen/202205/t20220512_3655336.html (accessed on 12 May 2022).

2. Zhou, J.; Li, E.; Yang, S.; Wang, M.; Shi, X.; Yao, S.; Mitri, H.S. Slope stability prediction for circular mode failure using gradient boosting machine approach based on an updated database of case histories. *Saf. Sci.* **2020**, *118*, 505–518. [CrossRef]
3. Yang, Y.; Thomas, N.S.; Li, N.; Xu, X.; Xu, P.; Xu, F.J. Adapting HLA-based co-simulation for interdependent infrastructure resilience management. *Automat. Constr.* **2023**, *150*, 104860. [CrossRef]
4. China News Service. Landslides and Subgrade Collapses on Mountain Slopes Have Blocked the Traffic on Several Expressways in Sichuan. Emergency Repairs Are Underway at Present. 2021. Available online: <https://www.chinanews.com.cn/sh/2021/10-05/9580233.shtml> (accessed on 5 October 2021).
5. Zhang, Y.; Wang, X. Study on causes of unsafe behaviors of construction workers based on DEMAREL-ISM-BN. *J. Saf. Sci. Technol.* **2020**, *16*, 110–116.
6. Zheng, Y.; Xie, Y.; Long, X. A comprehensive review of Bayesian statistics in natural hazards engineering. *Nat. Hazards* **2021**, *108*, 63–91. [CrossRef]
7. Hearn, G.J. Slope engineering for mountain roads. *Environ. Eng. Geosci.* **2013**, *19*, 204–205. [CrossRef]
8. Dahal, R.K. Regional-scale landslide activity and landslide susceptibility zonation in the Nepal Himalaya. *Environ. Earth Sci.* **2014**, *71*, 5145–5164. [CrossRef]
9. Chang, M.; Dou, X.; Tang, L. Risk assessment of multi-disaster in Mining Area of Guizhou, China. *Int. J. Disaster Risk Reduct.* **2022**, *78*, 103128. [CrossRef]
10. Yao, Y.; Fan, J.; Li, J. A Review of Advanced Soil Moisture Monitoring Techniques for Slope Stability Assessment. *Water* **2025**, *17*, 390. [CrossRef]
11. Zhang, X.; Li, Y.; Liu, Y.; Huang, Y.; Wang, Y.; Lu, Z. Characteristics and prevention mechanisms of artificial slope instability in the Chinese Loess Plateau. *Catena* **2021**, *207*, 105621. [CrossRef]
12. Van Asch, T.W.J.; Yu, B.; Hu, W. The development of a 1-D integrated hydro-mechanical model based on flume tests to unravel different hydrological triggering processes of debris flows. *Water* **2018**, *10*, 950. [CrossRef]
13. Zhu, Y.; Ishikawa, T.; Subramanian, S.S.; Luo, B. Simultaneous analysis of slope instabilities on a small catchment-scale using coupled surface and subsurface flows. *Eng. Geol.* **2020**, *275*, 105750. [CrossRef]
14. Zhao, H.; Ma, F.; Li, Z.; Guo, J.; Zhang, J. Geological hazards and protective measures of road slope in Himalayas Mountain area. *J. Eng. Geol.* **2022**, *30*, 656–671. [CrossRef]
15. Amarasinghe, M.P.; Kulathilaka, S.A.S.; Robert, D.J.; Zhou, A.; Jayathissa, H.A.G. Risk assessment and management of rainfall-induced landslides in tropical regions: A review. *Nat. Hazards* **2024**, *120*, 2179–2231. [CrossRef]
16. Zhang, R.; Ye, S.; Tao, H. Stability analysis of multistage homogeneous loess slopes by improved limit equilibrium method. *Rock Soil Mech.* **2021**, *42*, 813–825. [CrossRef]
17. Wu, Y.; Liu, T.; Wang, F.; Du, Z.; Lei, H. Stability assessment of the road cut slopes in the Xigeda mudstone considering long-term creep deterioration and suggestion for countermeasures: A case study of cut slopes along the Xichang-Panzhihua Expressway. *Chin. J. Geol. Hazard Control* **2024**, *35*, 56–66. [CrossRef]
18. Zhang, J.; He, P.; Xiao, J.; Xu, F. Risk assessment model of expansive soil slope stability based on Fuzzy-AHP method and its engineering application. *Geomat. Nat. Hazards Risk* **2018**, *9*, 389–402. [CrossRef]
19. Lin, A.; Wang, H.; Yan, B.; Dai, X.; Hu, X.; Zhao, X.; Wang, C. Quantitative risk assessment of the highway cutting slopes in adjacent transmission lines and design optimization for the reinforcement works. *Chin. J. Geol. Hazard Control* **2019**, *3*, 19–29. [CrossRef]
20. Niu, P.; Zhou, A.; Huang, H. Assessing model of highway slope stability based on optimized SVM. *Chin. Geol.* **2020**, *3*, 339–344. [CrossRef]
21. Yin, C.; Wang, Z.; Zhao, X. Spatial prediction of highway slope disasters based on convolution neural networks. *Nat. Hazards* **2022**, *113*, 813–831. [CrossRef]
22. Li, G.; Wu, X.; Han, J.C.; Li, B.; Huang, Y.; Wang, Y. Flood risk assessment by using an interpretative structural modeling-based Bayesian network approach (ISM-BN): An urban-level analysis of Shenzhen, China. *J. Environ. Manag.* **2023**, *329*, 117040. [CrossRef]
23. Zhang, T.; Su, H. A comprehensive evaluation method for dam service behavior based on Bayesian framework. *J. Changjiang River Sci. Res.* **2021**, *38*, 32–38. [CrossRef]
24. Tang, X.; Chen, A.; He, J. A modelling approach based on Bayesian networks for dam risk analysis: Integration of machine learning algorithm and domain knowledge. *Int. J. Disaster Risk Reduct.* **2022**, *71*, 102818. [CrossRef]
25. Zhang, J.; Wang, M.; Zhao, L.; Chen, M. Analysis of factors affecting prefabricated building quality based on ISM-BN. *Sustainability* **2023**, *15*, 9682. [CrossRef]
26. Shen, J.; Yin, Q.; Zhang, Y.; Zhang, C. Early Warning of Construction Period Risk of International Railway Projects Based on ISM-BN. *J. Shenyang Univ. (Nat. Sci.)* **2022**, *34*, 389–397. [CrossRef]
27. JTG 5150-2020; Technical Specification for Maintenance of Highway Subgrade. China Communications Press: Beijing, China, 2020.

28. Xu, J.; Liu, H.; Tang, L.; Zha, X.; Jiang, L. Analysis on typical diseases and prevention of service slopes of mountain expressways in Anhui Province. *Highway* **2022**, *67*, 419–425.
29. Lan, M.; Zhu, J.; Luo, X. Hybrid Bayesian network-based landslide risk assessment method for modeling risk for industrial facilities subjected to landslides. *Reliab. Eng. Syst. Saf.* **2021**, *215*, 107851. [CrossRef]
30. Ryoo, S.C.; Aydilek, A.H. Hydraulic Performance of Conventional and Conical Geotextile Units Filtering Two Highway Soils. *Geotech. Geol. Eng.* **2024**, *42*, 2623–2640. [CrossRef]
31. Zhao, Q.; Li, D.; Zhuo, M.; Guo, T.; Liao, Y.; Xie, Z. Effects of rainfall intensity and slope gradient on erosion characteristics of the red soil slope. *Stoch. Environ. Res. Risk A* **2015**, *29*, 609–621. [CrossRef]
32. Wu, L.Z.; Huang, R.Q.; Xu, Q.; Zhang, L.M.; Li, H.L. Analysis of physical testing of rainfall-induced soil slope failures. *Environ. Earth Sci.* **2015**, *73*, 8519–8531. [CrossRef]
33. Gupta, A.Z. Developing an Early Warning System of Highway Slope Failure Based on In-Situ Hydraulic Conductivity of Soil. Ph.D. Thesis, The University of Texas, Austin, TX, USA, 2023.
34. Cheng, Z.; Wang, Y.; Zhang, K.; Li, Q. Research on multi-level analysis method of highway slope maintenance decision during operation period. *Highway* **2022**, *67*, 337–343.
35. Zhao, H.; Tian, W.P.; Li, J.; Ma, B.C. Hazard zoning of trunk highway slope disasters: A case study in northern Shaanxi, China. *Bull. Eng. Geol. Environ.* **2018**, *77*, 1355–1364. [CrossRef]
36. Yang, W.; Zheng, M.; Li, Z.; Duan, L.W. Evaluation on coal measures soil slope instability risk based on combined weighted cloud model. *J. Highw. Transp. Res. Dev.* **2022**, *39*, 49–57. [CrossRef]
37. Huang, W.; Zhang, Y.; Kou, X.; Yin, D.; Mi, R.; Li, L. Railway dangerous goods transportation system risk analysis: An Interpretive Structural Modeling and Bayesian Network combining approach. *Reliab. Eng. Syst. Saf.* **2020**, *204*, 107220. [CrossRef]
38. GB/T 40112-2021; Specification for Risk Assessment of Geological Hazard. China Standards Press: Beijing, China, 2020.
39. Liu, Y.; Zhang, J.; Li, M.; Zhu, H.; Xiang, B. Fuzzy theory and SVM based Bayesian network assessment method for slope seismic instability scale. *Chin. J. Rock Mech. Eng.* **2019**, *38*, 2807–2815. [CrossRef]
40. Liu, Y.; Zhang, J.; Zhu, C.; Xiang, B.; Wang, D. Fuzzy-support vector machine geotechnical risk analysis method based on Bayesian network. *J. Mt. Sci.* **2019**, *16*, 1975–1985. [CrossRef]
41. Patt, A.G.; Schrag, D.P. Using specific language to describe risk and probability. *Clim. Chang.* **2003**, *61*, 17–30. [CrossRef]
42. Sun, J. Study on Collapse Risk and Stability Evaluation in Mining Construction of Mountain Tunnel. Ph.D. Thesis, Beijing Jiaotong University, Beijing, China, 2019.
43. Qiao, D.; Zhou, X.; Ye, X.; Tang, G.; Lu, L.; Ou, J. Security risk assessment of submerged floating tunnel based on fault tree and multistate fuzzy Bayesian network. *Ocean Coast. Manag.* **2024**, *258*, 107355. [CrossRef]
44. Langdalen, H.; Abrahamsen, B.A.; Selvik, J.T. On the importance of systems thinking when using the ALARP principle for risk management. *Reliab. Eng. Syst. Saf.* **2020**, *204*, 107222. [CrossRef]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Research on the Operation, Maintenance, and Parameters of Expressway Mechanical and Electrical Equipment Based on Markov Prediction

Xiaomin Dai ^{1,2}, Guojin Su ^{3,*}, Wei Tian ^{4,5,6} and Long Cheng ^{1,2}

¹ School of Transportation Engineering, Xinjiang University, Urumqi 830017, China; xmdai@xju.edu.cn (X.D.); chenglong@xju.edu.cn (L.C.)

² Xinjiang Key Laboratory of Green Construction and Maintenance of Transportation Infrastructure and Intelligent Traffic Control, Urumqi 830017, China

³ School of Business, Xinjiang University, Urumqi 830017, China

⁴ Xinjiang Jiaotou Engineering Technology Development Co., Ltd., Urumqi 830001, China; 13999181223@163.com

⁵ Xinjiang Transportation Investment (Group) Co., Ltd., Urumqi 830001, China

⁶ Engineering Technology and Transportation Industry in Arid Desert Areas, Urumqi 830001, China

* Correspondence: guojinsu@163.com; Tel.: +86-13276042552

Abstract: With the continuous progress of traffic technology and the continuous improvements in traffic infrastructure, the maintenance and management of highway mechanical and electrical equipment has become a key factor affecting highway operation efficiency. However, at present, most of the mechanical and electrical systems of expressways cannot monitor the equipment continuously in terms of operation and maintenance, and most of the equipment operation and maintenance stay only in the stage of equipment failure. In addition, there are many kinds of highway mechanical and electrical equipment, and there are significant differences in the levels of parameters, so the parameter levels of highway mechanical and electrical equipment cannot fully meet the operation requirements of the area. Therefore, based on the basic theory of the Markov chain and the concept of daily operation and maintenance, this paper constructs a multistate Markov fault prediction model considering maintenance. Based on the historical data, the model realizes the prediction of the equipment failure rate and the formulation of the optimal maintenance strategy for the equipment, taking video surveillance equipment as an example, and verifies the improvement in the value of the equipment under this strategy through the value engineering theory. Based on the prediction results, more reasonable technical parameters are customized for equipment with a high failure rate to improve the practicability and reliability of the mechanical and electrical equipment in the area.

Keywords: electromechanical equipment; fault prediction; Markov chain; value engineering

1. Introduction

The operation process of the electromechanical systems of expressways is uninterrupted for 24 h, which plays an important role in the smooth operation of the national highway network, and the occurrence of all kinds of equipment failures in the electromechanical system will directly affect the actual operation efficiency of the expressway, which means the accurate calculation of the equipment failure rate. Developing reliability-centric maintenance and replacement strategies is important to ensure service reliability and security [1]. For this situation, various prediction models such as Bayesian prediction [2,3],

the support vector machine model [4–6], and the logical regression model [7,8] have also developed rapidly and have been widely used in the field of engineering and technology. This kind of model can be used to calculate the probability of events according to some specific conditions, and the accuracy of the prediction results is more reliable. However, in the prediction of the fault types of mechanical and electrical equipment on expressways, there is still much room for improvement in the prediction and application of the above models. Limited by the operational environment of the mechanical and electrical equipment in the outfield of the highway, all kinds of environmental parameters and external stress conditions of the equipment during operation are multi-dimensional and dynamic, so it is difficult to monitor and record them in real time, which leads to the failure of the equipment. Its parameter changes are easy to miss. According to the current management strategy in most areas, maintenance operations are usually carried out only when errors are reported by the system or found manually after the failure of highway mechanical and electrical equipment. The lack of prior control has dramatically affected the reliability and remaining life of the equipment [9,10].

On this basis, some scholars have found that using the Markov process [11,12] to predict the failure rate can effectively avoid the problem that the early data are not applicable and the parameter records are missing when the equipment failure occurs. The prediction and analysis of equipment failure can be realized by constructing and analyzing the Markov fault model. Xinglong Wang [13] and other scholars established a Markov chain model by analyzing fault data and used this model to predict the occurrence frequency of different fault levels of ATCAS software in the next few months. Quan Yu [14] and others put forward a highway electromechanical equipment life prediction model based on the Markov chain, which provides a basis for expressway electromechanical equipment maintenance and renewal decision making. The new probabilistic modeling framework model for long-term fatigue reliability analysis of wind turbines proposed by Zhu D [15] makes innovative use of the Markov chain model and sub-simulation set and is an effective tool for the long-term fatigue reliability analysis of wind turbines and other structures. Qingzhu L. et al. [16] constructed a simplified reliability evaluation model of a complex security system based on the Markov state. The system reliability deviation between the simplified and complete models is less than 0.0021%, proving the simplified model's effectiveness. Haghighi O [17] proposed a ten-state extended Markov model of CVT (Continuously Variable Transmission) and proved by simulation that the compensating reactors in the second capacitance group and EMUs (Electric Multiple Units) subsystem play an important role in the performance of the CVT. Yang Yi [18] and others established a semi-Markov model (UPSM) of uncertain seepage to describe the physical characteristics of traffic accidents considering random and cognitive uncertainty. In summary, using Markov theory for related prediction and analysis can significantly improve the stability of equipment operation and maintenance from the external operation and maintenance.

However, in addition to the external operating environment, the parameter decay of the equipment itself will also affect its reliability. When electromechanical equipment is subjected to external stress or affected by the environment, its own performance parameters may be degraded, such as the reduction in coil detection accuracy, the decline in waterproof and dustproof grade, and so on. These changes may lead to weakening of the equipment's function, shortening of its life, and even failure, thus increasing the risk of equipment failure and reducing the reliability and safety of electromechanical equipment. By studying the changes in parameters in the internal decay process of the equipment, the reliability of the equipment can be effectively analyzed. Then, the corresponding prevention and parameter optimization measures can be put forward to effectively reduce the equipment's failure rate and improve the equipment's reliability, prolong the service life of the equip-

ment, reduce the maintenance cost, and improve the use efficiency and economic benefit. Xiaogang Zhang [19] and other scholars use equipment performance parameters such as equipment utilization, manufacturing capacity, and productivity to evaluate the reliability of flexible manufacturing cells. Chang Sun [20] uses a non-dominant sorting genetic algorithm (NSGA) and entropy weight-based optimal selection method (TOPSIS) to optimize the parameters, which provides a solution for soil temperature reduction and low-cost operation in cold and arid areas in the application of ground-source heat pumps. Li X Y [21], combining the data characteristics of the equipment accelerated decay experiment and considering the cognitive uncertainty, proposed a stochastic fuzzy accelerated decay model and its corresponding statistical analysis method. Li N [22] proposed a method to describe the equipment performance decay process by using the general expression based on the life and state correlation model, and the general expression of the stochastic process solved the probability density function.

This study creatively combines the external operation and maintenance strategy of highway mechanical and electrical equipment with the internal technical parameters of the equipment, formulates its operation and maintenance strategy from the point of view of equipment maintenance, and starts with the shortcomings of the operation and maintenance strategy to improve the parameter level of the equipment itself. Firstly, a multistate Markov model considering maintenance is constructed based on the basic theory of the Markov chain. Based on the fault performance in the historical operation and maintenance data of electromechanical equipment, the fault state of the equipment is classified, and the maintenance factors of the equipment's operation and maintenance are taken into consideration. By predicting the future fault state of electromechanical equipment, the purpose of this model is to realize the differential operation and maintenance strategy based on the external information of the equipment. In addition, aiming at the typical equipment with high failure rates, this study customizes the technical parameters to adapt to the environment of the equipment from the inside in order to improve the reliability of the equipment. The equipment operation and maintenance and parameter research involved in the study comprehensively considers the external operation and maintenance environment and internal quality factors of the equipment, which can essentially enhance the use value of highway mechanical and electrical equipment. The specific research ideas are shown in Figure 1.

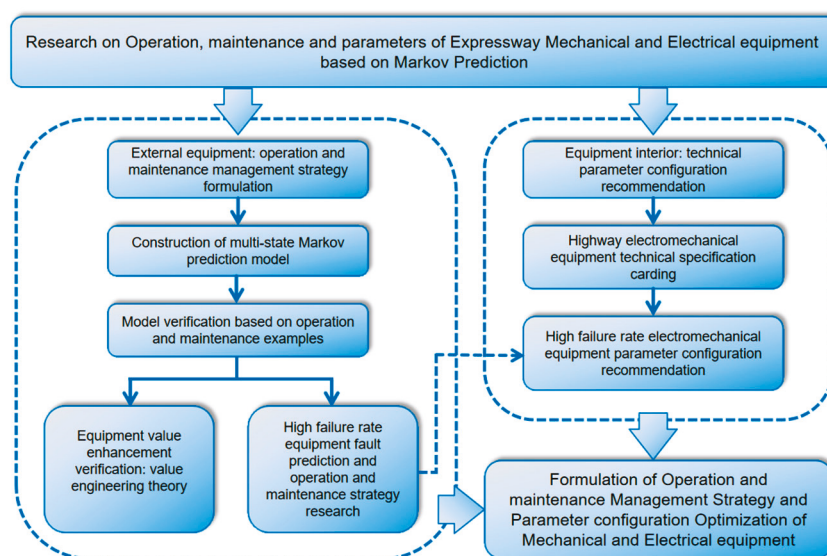


Figure 1. Research technology roadmap.

2. Research Methodology

2.1. Multistate Markov Chain Model

A Markov chain is a discrete event random process that has no memory. The future state depends only on the current state and can be predicted only by the current state. The Markov chain plays a vital role in many fields, such as industry, finance, social science, and communication. Its application value is significant, and it supports the development of various fields. This study first discusses the basic theory of the Markov chain. Then, a Markov fault prediction model of a multistate electromechanical system considering maintenance is constructed based on this theory.

2.1.1. Conceptual Description of the Markov Chain

The discrete state space $E = \{S_1, S_2, S_3, \dots\}$ and the random process $\{X(t)/X(t) \in E, t > 0\}$ on the discrete time parameters $0 \leq t_1 < t_2 < t_3 < \dots < t_n$ exist at any time. From the definition of conditional probability, the state probability t_n at different times can be obtained. The specific formula is expressed in Formula (1).

$$P[X(t_n)/X(t_1) \cdots X(t_{n-1})] = S_n[X(t_1) = S_1 \cdots X(t_{n-1}) = S_{n-1}] \quad (1)$$

If the random process is consistent with Formula (2), then the random process accords with the characteristic of no memory, and the random process is called a Markov chain.

$$P[X(t_n)/X(t_1) \cdots X(t_{n-1})] = P[X(t_1) = X(t_n)/X(t_{n-1})] \quad (2)$$

Assuming that the stochastic process is a homogeneous Markov process (3), the state statistics of the Markov chain determine the conditional probability $P_{ij}(\Delta t)$, and $P_{ij}(\Delta t)$ is the conditional probability of the transition from state S_i to state S_j in Δt . The determination of conditional probability is one of the important links in the application of Markov chain theory.

$$P[X(t + \Delta t) = S_j/X(t) = S_i] = P[X(\Delta t) = S_j/X(0) = S_i] = P_{ij}(\Delta t) \quad (3)$$

For the state space composed of n states with the same time interval of $t_1, t_2, t_3, \dots, t_n$, the state transition probability can be expressed in the form of a matrix, as shown in Formula (4).

$$P = P_{ij} = \begin{pmatrix} P_{11} & P_{12} & \cdots & P_{1n} \\ P_{21} & P_{22} & \cdots & P_{2n} \\ \vdots & \vdots & & \vdots \\ P_{n1} & P_{n2} & \cdots & P_{nn} \end{pmatrix}, P_{ij} \geq 0, \sum_{j=1}^n P_{ij} = 1; \quad (4)$$

Therefore, the state transition probability matrix P that meets the precision requirements can be obtained by collecting the state transition data between S_m and S_{m+1} . At the same time, whenever a set of data samples is added, the probability value of only one row of matrix P will change, and the number of rows corresponds to the state of the system at the time of t_m . Theoretically, the statistical value of matrix P will gradually approach its real value when the number of samples tends to infinity.

2.1.2. Multistate Markov Chain Model of Electromechanical Systems

The Markov chain, as a mathematical method, can play a role in equipment fault prediction and maintenance planning [23]. The key is to describe the transition between states and its corresponding probability [24]. In this study, the basic process of the Markov chain is applied to the field of highway mechanical and electrical equipment management,

and the equipment state is divided into normal operation state, slight fault state, general fault state, and serious fault state. By considering the situation of daily maintenance and analyzing the transfer probability between equipment states, a Markov fault prediction model of multistate electromechanical systems, considering maintenance, is proposed. This model is used to analyze the probability of occurrence in various states of the electromechanical system in the future and to propose maintenance strategies.

In the fault prediction of the mechanical and electrical equipment of expressways in the outer field, the Markov model of regular operation and faults can be used to simulate the operation process of the highways' mechanical and electrical equipment. This model is shown in Figure 2, where S_0 represents the normal running state of the equipment; S_F represents the failure status of the equipment; and λ and μ represent the failure rate and repair completion rate of the equipment, respectively.

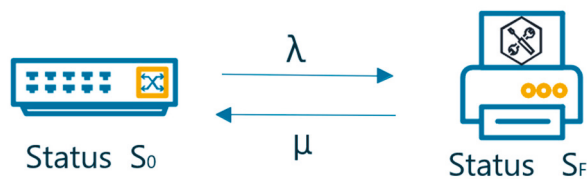


Figure 2. Markov chain model of two states of highway mechanical and electrical equipment.

According to the abovementioned model, the fundamental failure rate of the electromechanical system can be calculated. However, the process of operation and maintenance of highway mechanical and electrical equipment is not limited to simple regular operation and fault state. It may include parts maintenance (such as minor failure), overall equipment replacement (such as severe failure), and other situations. This study extends the Markov chain model of two basic states to a multistate model, as shown in Figure 3, to better reflect the operation and maintenance status of highway mechanical and electrical equipment. In the absence of any maintenance policy, the equipment state will follow the Markov chain transfer from S_0 to S_F until a severe failure occurs; that is, it enters the S_F state. Measures such as maintenance and replacement are needed to restore the equipment to its intact state, S_0 .

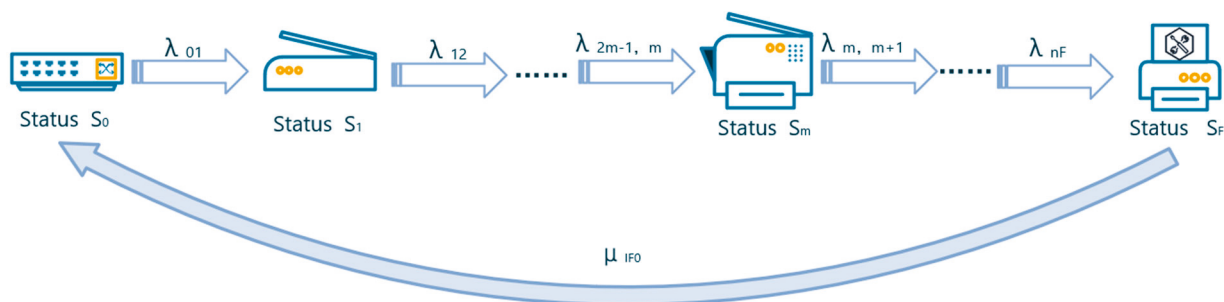


Figure 3. Multistate Markov chain model of electromechanical equipment.

In Figure 4, S_0 represents the normal state of electromechanical equipment, S_1 and S_2 represent the slight and general fault states of electromechanical equipment, and S_F represents the serious fault state of highway electromechanical equipment, which requires the replacement of the main parts of the faulty equipment or the replacement of the entire equipment to restore the regular operation of the electromechanical system. I_0 , I_1 , and I_2 represent the equipment in the state of inspection. In addition, in the state classification of equipment faults, safety elements need to be put in the first place, and if equipment failures can affect highway traffic safety, then no matter what state the equipment is in, it should be regarded as a serious fault state, so as to warn the staff to deal with it immediately. Furthermore, λ_{01} , λ_{12} , λ_{2F} , λ_I , μ_{I0} , μ_{I1} , μ_{I2} , and μ_{IF0} are the state transfer rates of

mechanical and electrical equipment. This study assumes that two kinds of maintenance methods are adopted, minor maintenance and deep maintenance. The process from I_1 to S_0 is the process of minor maintenance, and μ_{I10} represents the completion rate of minor maintenance. The process from I_2 to S_0 is the process of deep maintenance, and μ_{I20} represents the completion rate of deep maintenance.

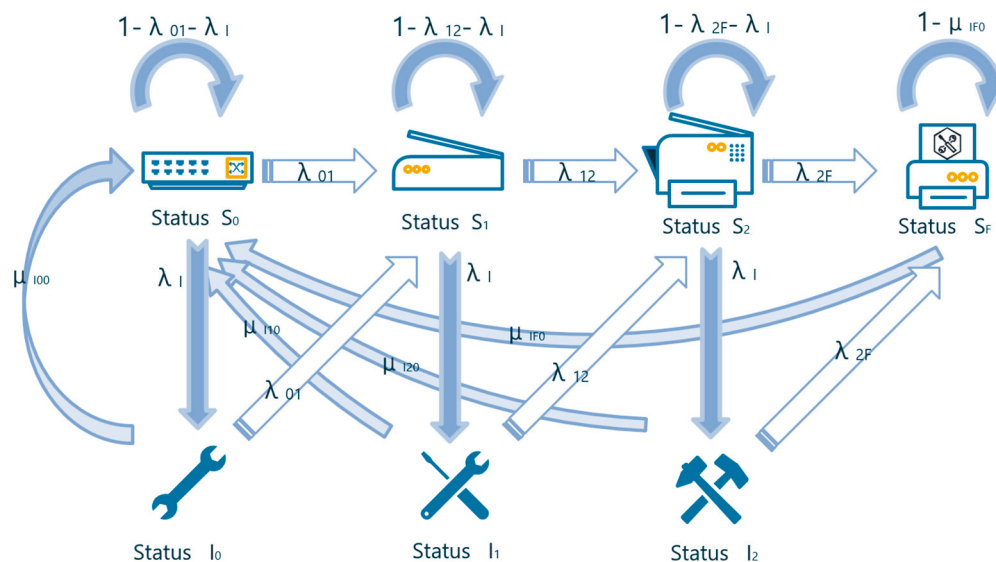


Figure 4. Multistate Markov chain model of electromechanical system considering the maintenance strategy.

Under the setting of this multistate Markov model, the state transfer process of highway mechanical and electrical equipment during operation can be described as follows: maintenance personnel regularly inspect the mechanical and electrical equipment according to the frequency of λ_I . If the equipment is found to be as expected, it will not be overhauled. If the equipment is in the abnormal state S_1 , then slight maintenance measures will be taken to make the equipment return to the normal state. If the equipment is in the serious fault state S_2 , then deep maintenance measures will be taken to make the equipment return to the normal state. After considering the maintenance strategy, the time for the equipment to reach the serious fault state will be delayed, and the reliability of all electromechanical equipment will be improved to a certain extent.

$$P = \begin{bmatrix} 1 - (\lambda_{01} + \lambda_I) & \lambda_{01} & 0 & 0 & \lambda_I & 0 & 0 \\ 0 & 1 - (\lambda_{12} + \lambda_I) & \lambda_{12} & 0 & 0 & \lambda_I & 0 \\ 0 & 0 & 1 - (\lambda_{2F} + \lambda_I) & \lambda_{2F} & 0 & 0 & \lambda_I \\ 0 & 0 & 0 & 1 - \mu_{F0} & 0 & 0 & 0 \\ \mu_{I00} & \lambda_{01} & 0 & 0 & 1 - (\lambda_{01} + \mu_{I00}) & 0 & 0 \\ \mu_{I10} & 0 & \lambda_{12} & 0 & 0 & 1 - (\lambda_{12} + \mu_{I10}) & 0 \\ \mu_{I20} & 0 & 0 & \lambda_{2F} & 0 & 0 & 1 - (\lambda_{2F} + \mu_{I20}) \end{bmatrix} \quad (5)$$

Based on the state transition principle of the Markov chain model, the state transition probability matrix P can be obtained, as shown in Formula (5). The state probability of an electromechanical system is written as a column vector $X = [X_0, X_1, X_2, X_F, X_{I0}, X_{I1}, X_{I2}]$. It is combined with the Markov state transition matrix, operation and maintenance records analysis, and failure rate data. The initial failure rate and transfer matrix are obtained by solving the MATLAB program, which can be used to determine the occurrence probability of each equipment state.

2.2. Value Engineering Theory

In this study, in order to verify the effectiveness of the operation and maintenance strategy in promoting the value of equipment, according to the value engineering theory [25,26] combined with the established optimal maintenance strategy, and comprehensively considering the cost factors such as time cost and human resources involved in maintenance operations, an accurate calculation is carried out, and the value coefficient of the equipment is obtained. In mechanical and electrical projects, the value of highway mechanical and electrical equipment is usually regarded as one of the important means to evaluate the performance-to-price ratio of mechanical and electrical equipment. Specifically, the value coefficient of the target is determined as the inverse relationship between function consumption and life cycle cost. This method can obtain an objective performance-to-price index to guide project decision making and cost control through the comprehensive analysis of the function consumption and operation and maintenance cost using the basic formula of value engineering (6):

$$V = \frac{F}{C} \quad (6)$$

In the formula, v is value, meaning value coefficient; F is function, meaning function coefficient; C is cost, meaning cost coefficient.

The core feature of value engineering theory is to pursue the maximum function or economic benefit with as little cost as possible. As the function ratio to target cost, the value coefficient of mechanical and electrical equipment can effectively reflect its target value. Take the typical highway mechanical and electrical equipment on the market. The core criterion of its importance and value coefficient lies in whether it has high quality and low price characteristics. “High quality” mainly focuses on the functional attributes of the equipment, that is, whether it can meet the expected use needs and performance standards, while “low price” mainly focuses on the cost composition of the equipment, including its production costs, label costs, and other economic costs. Simultaneously, in the daily operation and maintenance of the mechanical and electrical system, we must pursue the lowest possible maintenance cost to ensure the system’s stable operation and improve overall economic and operational benefits.

3. Case-Based Model Validation

The research must first verify its effectiveness for the multistate Markov fault prediction model considering maintenance factors. This section will select the historical operation and maintenance data of the corresponding sections and make a holistic analysis of the running state of the expressway electromechanical system, which can be used to preliminarily determine that the application of the model can effectively improve the expected state probability of the overall operation of the electromechanical system.

3.1. Data Carding

This study predicts the overall fault condition of the electromechanical system based on the electromechanical operation and maintenance work order summary account of the Urumqi Expressway on Line G3003 in Xinjiang in October 2023, which shows the effectiveness of the multistate Markov chain model considering maintenance.

A total of 4289 mechanical and electrical equipment sets are present in the Urumqi Round City Expressway (East) and Xiaocaohu Urumqi Expressway (G30). By combing the data in the fault maintenance list of the abovementioned sections in October, 4289 valid fault operation and maintenance records are obtained, including 42 slight fault status S_1 data, 81 serious fault status S_2 data, and 8 equipment or parts status S_F data. Maintenance record data are shown in Table 1.

Table 1. Maintenance record data.

Status		Quantity	
Minor fault status: S_1	Completed	Field maintenance	36
		Remote processing	4
		Unfinished	2
General failure status: S_2	Completed		78
	Unfinished		3
Serious failure status: S_F			8
			8

3.2. Running State Prediction

After combing the overall data of the two road sections, the data can be imported into the model to predict the state occurrence probability and formulate the overall operation and maintenance strategy.

By analyzing the number of times each degradation state occurs and the completion of manual operation and maintenance, and considering the maintenance code specific to the mechanical and electrical facilities industry, the initial state transfer probability of the electromechanical system can be calculated as $\lambda_{01} = 0.98\%$, $\lambda_{12} = 4.76\%$, $\lambda_{2F} = 3.70\%$. The repair completion rates of minor fault state S_0 , general fault state S_2 , and serious fault state S_F are 95.24%, 96.30%, and 100%, respectively.

First, the study calculates the situation when the maintenance strategy is not considered. In this case, the probability of I_0 , I_1 , and I_2 is 0, the total number of known devices is 4289, and the number of fault data is 131. Combined with the numbers of slight fault state, general fault state, and serious fault state in Table 1, the steady-state probability of each state of electromechanical system follows Equation (7):

$$X = [0.9695, 0.0098, 0.0189, 0.0018, 0, 0, 0] \quad (7)$$

Combined with Figure 4 and Table 1, the state transition matrix P can be obtained, as shown in matrix (8).

$$P = \begin{bmatrix} 0.9902 & 0.0098 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.9524 & 0.0476 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.9630 & 0.0370 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \\ 0.9902 & 0.0098 & 0 & 0 & 0 & 0 & 0 \\ 0.9524 & 0 & 0.0476 & 0 & 0 & 0 & 0 \\ 0.9630 & 0 & 0 & 0.0370 & 0 & 0 & 0 \end{bmatrix} \quad (8)$$

Taking Formula (7) as the initial state and iterating Formula (8) to the steady state with MATLAB R2022a, the steady-state probability calculation results of each state of the electromechanical system without considering overmaintenance strategy can be obtained, as shown in Equation (9).

$$\Pi = [0.6846, 0.1049, 0.1813, 0.0070, 0, 0, 0] \quad (9)$$

Then, the situation considering the maintenance strategy is calculated. If the initial frequency of manual inspection in the maintenance strategy is set to 1/month (the working time of a working day is 8 h, and a month is calculated according to 30 working days) and analyzed, then the initial state transition matrix (10) of the electromechanical system

can be obtained. The state transition model of the multistate Markov process considering maintenance is shown in Figure 5.

$$P = \begin{bmatrix} 0.9569 & 0.0153 & 0 & 0 & 0.0333 & 0 & 0 \\ 0 & 0.9191 & 0.0526 & 0 & 0 & 0.0333 & 0 \\ 0 & 0 & 0.9297 & 0.0555 & 0 & 0 & 0.0333 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \\ 0.9847 & 0.0153 & 0 & 0 & 0 & 0 & 0 \\ 0.9474 & 0 & 0.0526 & 0 & 0 & 0 & 0 \\ 0.9445 & 0 & 0 & 0.0555 & 0 & 0 & 0 \end{bmatrix} \quad (10)$$

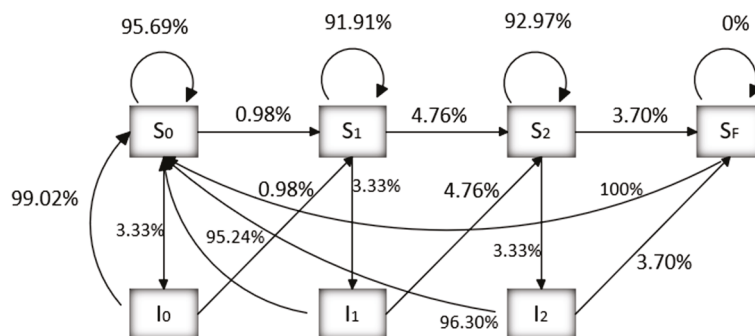


Figure 5. Diagram of the state transition probability model considering maintenance.

By extracting the state transition matrix of the electromechanical system and the probability initial values of each state into MATLAB R2022a to iteratively calculate to the steady state, the limit distribution of the Markov chain fault prediction model considering maintenance can be obtained, as shown in Equation (11):

$$\Pi = [0.7936, 0.0993, 0.0695, 0.0028, 0.0264, 0.0033, 0.0051] \quad (11)$$

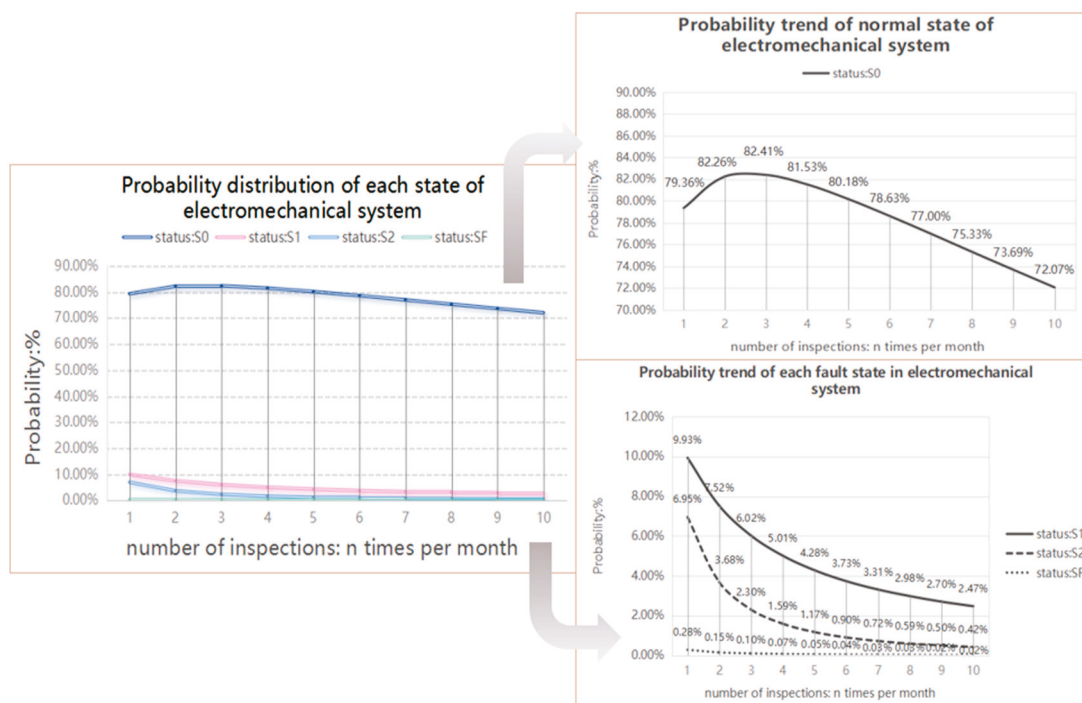
In the operation and maintenance process of the electromechanical system, regular manual inspection can help ensure the normal operation of the equipment, reduce maintenance costs, improve production efficiency, and increase the life of the equipment. Thus, this study gradually increases the frequency of manual inspection of the electromechanical system to formulate a better operation strategy. The aim is to ensure that all kinds of equipment for the electromechanical system can be kept better in the normal working state to improve the equipment's reliability and the electromechanical system's stability.

After the frequency adjustment and calculation of manual inspection, this study proposes to gradually increase the manual inspection frequency of the expressway electromechanical system from once a month to 10 times a month, and different frequencies are introduced into the multistate Markov model for testing. The stationary distribution of the electromechanical system's corresponding multistate Markov fault state is summarized in Table 2, and the trend change in the corresponding state is shown in Figure 6.

Figure 6 shows that the probability of mild (S_1), general (S_2), and severe (S_F) failure statuses gradually decreases with the increase in maintenance frequency and the rise in inspection time. The occurrence probability of a normal operation state (S_0) does not maintain a steady upward trend when the inspection frequency is set to three times per month. The occurrence probability of the normal state (S_0) reaches a peak of 82.41%, an increase of 13.95% compared with 68.46% of the expected state probability of the electromechanical system under manual intervention.

Table 2. State distribution corresponding to manual inspection ratings 1–10.

Number of Inspections: n Times per Month		1	2	3	4	5
State occurrence probability	Status: S_0	79.36%	82.26%	82.41%	81.53%	80.18%
	Status: S_1	9.93%	7.52%	6.02%	5.01%	4.28%
	Status: S_2	6.95%	3.68%	2.30%	1.59%	1.17%
	Status: S_F	0.28%	0.15%	0.10%	0.07%	0.05%
Number of inspections: n times per month		6	7	8	9	10
State occurrence probability	Status: S_0	78.63%	77.00%	75.33%	73.69%	72.07%
	Status: S_1	3.73%	3.31%	2.98%	2.70%	2.47%
	Status: S_2	0.90%	0.72%	0.59%	0.50%	0.42%
	Status: S_F	0.04%	0.03%	0.03%	0.02%	0.02%

**Figure 6.** Fault state prediction trend chart of electromechanical systems.

In summary, the equipment inspection requires a lot of human and material resources. If the maintenance frequency is too high, the inspection cost, including workforce, time, and maintenance costs, will increase, and the low inspection frequency may affect the system's stability. By optimizing the inspection frequency and keeping it within an appropriate range, the cost can be reduced while ensuring the regular operation of the electromechanical system. Because of the three fault states of S_1 , S_2 , and S_F , three times a month, that is, three working days a month, can be taken to conduct a comprehensive investigation of the electromechanical system. This scheme not only can effectively ensure the normal operation rate of the electromechanical system but also save a certain amount of resource investment and maximize benefits.

4. Formulation of an Operation and Maintenance Strategy for Equipment with a High Failure Rate

In the previous section, through the systematic prediction and analysis of the operation and maintenance data of highway mechanical and electrical equipment in two specific sections in Xinjiang, we preliminarily verified the effectiveness of the Markov chain's basic theory in optimizing the electromechanical system's maintenance strategy. However, given the significant heterogeneity of all kinds of equipment in highway electromechanical systems in terms of function, operating environment, and application scene, there is still much room for improvement in the overall fault prediction and analysis of expressway electromechanical equipment in the previous section. Therefore, in this section, the research focuses on the equipment with a high failure rate by predicting its failure probability and putting forward the corresponding operation and maintenance strategy to improve further the reliability of the equipment and prolong its service life.

4.1. Data Processing and Fault Prediction

There are 5196 sets of mechanical and electrical equipment in three sections of the Xinjiang expressway: Urumqi Round City Expressway (East), Xiaocaohu Urumqi Expressway (G30), and the Jingxin Expressway Dahuangshan–Urumqi Section Reconstruction and Expansion Project (G7). Combining the operation and maintenance data of the above sections in October 2023, 5196 effective fault operation and maintenance records are obtained. In this record, the equipment failure of the expressway electromechanical system is mainly shown in the related equipment of the toll collection system and is summarized in Table 3.

Table 3. Overview of the main failures of the toll system.

Type of Fault Equipment	Number of Fault Records	Proportion of Quantity	Fault Performance
Video surveillance equipment	29	17.6%	The picture is not clear; identify fault, etc.
Automatic railing machine	20	12.1%	Bar rise/fall fault; railing arm fracture; railing machine does not have power, etc.
Weighing equipment	15	9.1%	Axis identification error; weighing platform collapse; weighing platform overweight; no axis information, etc.

The operation and maintenance records show that video surveillance equipment, weighing equipment, automatic railing machines, and license plate recognition instruments have a higher failure rate in the two sections. Based on this, this section takes video surveillance as an example to explore its optimal external operation and maintenance strategy.

4.1.1. Data Carding and Preprocessing

In the three sections of Urumqi Round City Expressway (East) (G3003), Xiaocaohu Urumqi Expressway (G30), and the reconstruction and expansion project of Dahuangshan–Urumqi Section of Jingxin Expressway (G7), there are 5196 sets of expressway mechanical and electrical equipment. Among them are 981 pieces of video surveillance equipment, and the amount of fault recording data related to this kind of equipment is 29. In the above three road sections, the fault recording data are as high as 17.6%, which belongs to the equipment with high fault incidence.

Among these kinds of equipment's fault records are 27 minor fault status records, 2 general fault status records, and 0 serious fault status records, of which 2 minor fault repair works have not been completed. In addition, if the equipment failure state is not transferred to a more serious state, the transfer rate is set to 0.1%, the corresponding

maintenance state transfer rate is 0.1%, and the repair completion rate is 99.9%. In the case of severe failure, the repair completion rate is 100%, which indicates that the equipment is replaced or the core components are replaced to ensure that the equipment is completely repaired.

Based on the above data, the initial state transition probability of video surveillance equipment is $\lambda_{01} = 2.75\%$, $\lambda_{12} = 7.40\%$, $\lambda_{2F} = 0.10\%$. The repair completion rate of the minor fault state S_1 is 92.59%, the general fault state S_2 is 99.90%, and the serious fault state S_F is 100%.

As in Section 3.2, combined with the number of equipment items, the amount of fault data, and the state transfer rate in the selected section, the initial probability and transfer rate matrices of each state of the video surveillance equipment are shown in Equation (12) and matrix (13):

$$X = [0.8985, 0.0275, 0.0730, 0.0010, 0, 0, 0] \quad (12)$$

$$P = \begin{bmatrix} 0.9725 & 0.0275 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.9270 & 0.0730 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.9990 & 0.0010 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0.9725 & 0.0275 & 0 & 0 & 0 & 0 & 0 \\ 0.9270 & 0 & 0.0730 & 0 & 0 & 0 & 0 \\ 0.9990 & 0 & 0 & 0.001 & 0 & 0 & 0 \end{bmatrix} \quad (13)$$

4.1.2. Operation and Maintenance Strategy Formulation

Taking Formula (12) as the initial state, Formula (13) is calculated with MATLAB R2022a to calculate the steady-state probability of each failure state of the video surveillance equipment without considering the overmaintenance strategy, as shown in Equation (14):

$$\Pi = [0.0346, 0.0130, 0.9514, 0.0010, 0, 0, 0] \quad (14)$$

It can be seen that without implementing the maintenance strategy, only 3.46% of the video surveillance equipment in these sections can remain in a normal state over time. In comparison, 95.14% of the video surveillance equipment without regular maintenance will begin to have problems such as unstable signals, occasional stutters, and inability to transmit pictures, which will significantly affect the regular operation of the mechanical and electrical system and bring great trouble to the monitoring personnel.

The corresponding time and resources must be invested when considering the maintenance situation. Subject to the standard working hours system, an individual staff member's total statutory working hours are approximately 176 h in a natural month. Video surveillance equipment can maintain the most efficient and regular operation by strictly following the maintenance strategy. This study adopts a step-by-step method, taking one person-hour (1 h) as the time unit and gradually increasing the frequency of maintenance until the most ideal maintenance measures are found to improve the operation efficiency of the equipment. According to different fault types and state changes, the video surveillance equipment's initial fault transfer rate can be obtained, from the normal to the fault state. As shown in Figure 7, the corresponding maintenance strategy solution matrix (15) is written as follows:

$$P = \begin{bmatrix} 1 - (\lambda_{01} + \lambda_I) & 0.0275 & 0 & 0 & \lambda_I & 0 & 0 \\ 0 & 1 - (\lambda_{12} + \lambda_I) & 0.073 & 0 & 0 & \lambda_I & 0 \\ 0 & 0 & 1 - (\lambda_{2F} + \lambda_I) & 0.001 & 0 & 0 & \lambda_I \\ 1 & 0 & 0 & 1 - \mu_{F0} & 0 & 0 & 0 \\ 0.9725 & 0.0275 & 0 & 0 & 1 - (\lambda_{01} + \mu_{I00}) & 0 & 0 \\ 0.927 & 0 & 0.0730 & 0 & 0 & 1 - (\lambda_{12} + \mu_{I10}) & 0 \\ 0.999 & 0 & 0 & 0.001 & 0 & 0 & 1 - (\lambda_{2F} + \mu_{I20}) \end{bmatrix} \quad (15)$$

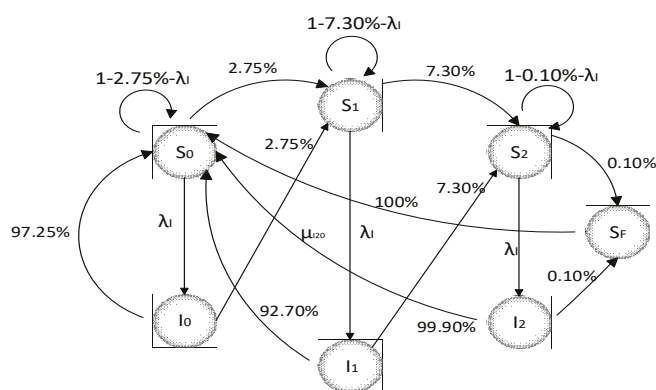


Figure 7. Initial state transfer rate diagram of video surveillance equipment.

The ratio of monthly maintenance hours to monthly legal working hours is set as the inspection frequency of the maintenance strategy. Among them, the daily legal working hours T_{legal} is calculated according to the “measures for the preparation of budgetary estimates for Highway Construction projects” (JTG3830-2018), and the number of monthly working days is 22. That is,

$$\lambda_I = \frac{t}{T_{legal}} \quad (16)$$

By substituting Formula (16) into the strategic maintenance matrix of the video surveillance equipment and solving its steady-state value, it can be concluded that when the monthly maintenance time is 29 person-hours, that is, when the maintenance frequency is 0.1648, the standard running state value of the long-term operation of the video surveillance equipment reaches the peak, which is 71.31%. The trend of each state of this kind of equipment changing with the maintenance time is shown in Figure 8.

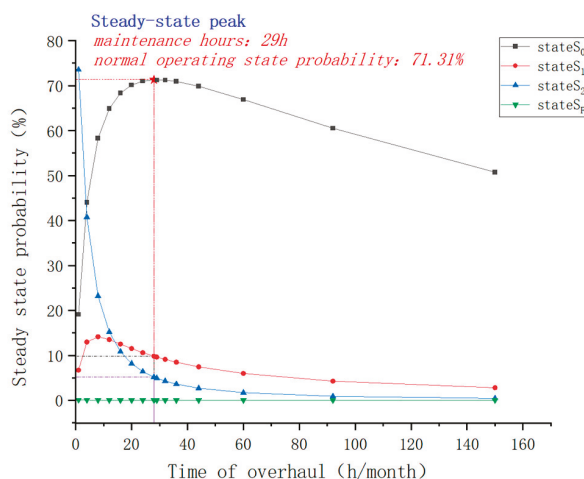


Figure 8. Trend chart of the steady-state probability of video surveillance equipment.

The state transition matrix under the inspection policy is shown in Table 4.

Table 4. Matrix table of optimal maintenance strategy.

Transfer Probability	λ_{01}	λ_{12}	λ_{2F}	μ_{100}	μ_{110}	μ_{120}	μ_{F0}
	0.0275	0.073	0.001	0.9725	0.927	0.999	1
State	S_0	S_1	S_2	S_F	I_0	I_1	I_2
S_0	0.8077	0.0275	0	0	0.1648	0	0
S_1	0	0.7622	0.073	0	0	0.1648	0
S_2	0	0	0.8342	0.001	0	0	0.1648
S_F	1	0	0	0	0	0	0
I_0	0.9725	0.0275	0	0	0	0	0
I_1	0.927	0	0.073	0	0	0	0
I_2	0.999	0	0	0.001	0	0	0

The steady-state probability of each state under the maintenance strategy is as follows (Equation (17)):

$$\Pi = [0.7131, 0.0961, 0.0493, 0.0010, 0.1175, 0.0158, 0.0081] \quad (17)$$

Figure 8 shows that, in theory, when implementing the strategy of 29 person-hours per month for video surveillance equipment, not only can the operation stability of the video surveillance equipment reach the highest level under long-term operation, but also the probability of general failure (state S_2) of this kind of equipment can be significantly reduced. By reducing in-depth maintenance measures, the maintenance cost of video surveillance equipment is significantly saved, thus saving the cost of operation and maintenance of this kind of equipment and increasing the value of the equipment.

4.2. Verification of Equipment Value Improvement

In the previous section, this study obtained the external optimal operation and maintenance strategy, taking video surveillance as an example. This section introduces the theory of value engineering to verify the effectiveness of the operation and maintenance strategy to enhance the value of video surveillance equipment. The study comprehensively considers the cost factors such as time and human resources in the maintenance operation. The value coefficient of 10 sets of video surveillance equipment in the 3 sections involved in the study is calculated to verify the improvement of the value of this kind of equipment according to the basic formula of value engineering, $V = \frac{F}{C}$. The research still takes video surveillance equipment as an example. It sets the function coefficient and cost coefficient of the equipment according to the corresponding national standards of the area where the electromechanical equipment belongs and the norms of the transportation and machinery industry to calculate the improvement in the value coefficient of the equipment before and after the implementation of the optimal operation and maintenance strategy and verifies the effectiveness of the strategy.

4.2.1. Function Coefficient

According to the Highway Engineering Budget quota (JTGT 3832-2018), the quota for the toll plaza camera is CNY 17094.02, which is set as the functional value generated during the specified service life of this kind of equipment, that is, Fair 17094.02.

4.2.2. Cost Coefficient

Assuming that the everyday operation service life of each set of equipment is 3 years, according to the "Highway Engineering Estimation Index" (JTGT/T3821-2018)

and “Highway Engineering Budget Quota” (JTG/T3832-2018), the manual maintenance cost incurred during maintenance is calculated at 106.28/person-day; the material fees and machinery and shift costs incurred during maintenance are shown in Table 5, in which minor and sporadic materials and small construction machinery are not all listed and are included in “fees for other materials” and “fees for the use of small machines and tools”, respectively. During the inspection and repair of the cameras in the toll plaza, it is necessary to arrange trucks within 4 t and hydraulic lifts within 300 kg to operate. According to the Highway Construction Machinery shift fee quota (JTG/T3833-2018), the cost for each shift of trucks within 4 t is CNY 29.28 per shift for hydraulic lifts within 79.56300 kg. Based on this, it can be calculated that the total cost of each working day for horizontal and vertical transportation is CNY 8.3 when overhauling the video surveillance equipment in the toll square.

Table 5. Resource consumption and basic pricing table for installation and debugging of each set of video surveillance equipment.

Serial Number	Name	Unit	Video Surveillance Equipment				
			Toll Booth	Toll Island	Toll Plaza	Tunnel	Indoor
			1	2	3	4	5
1	Fees for labor	Working day	3.8	4.8	9	3.6	3.6
2	Bolts	Kg	—	—	2.9	—	—
3	Expansion bolts	Set group	4.1	10.2	10.2	8.2	8.2
4	Other materials	CNY	1.9	2.5	4.1	1.9	1.9
5	Trucks less than 4 t	Shift group	—	—	0.88	0.22	0.18
6	Hydraulic lifts within 300 kg	Shift group	—	—	0.16	—	—
7	Small tools	CNY	12.2	23.6	62.7	13.1	13.1
8	Basic pricing	CNY	353	585	1529	540	522

According to the “Budget preparation method for Highway Construction projects” (JTG3830-2018), the working hours per comprehensive working day are calculated according to 8 h. Based on this, the minor maintenance cost (c_1), the general maintenance cost (c_2), and the deep maintenance cost (c_3) are 13.28, 15.71, and 21.24, respectively. The cost of staffing and tool materials generated by the inspection strategy is the average value of the three maintenance modes, $c_I = 16.74$.

4.2.3. Value Coefficient Calculation

Assuming that the occurrence probability of each state of the camera in the toll square reaches a stable state after 3 years of service, the annual cost of not implementing the maintenance strategy can be calculated as $C_n = 130,812.83$, and the annual maintenance cost of implementing the maintenance strategy as $C_s = 18,725.39$. Then, the value coefficient of video surveillance equipment without the maintenance strategy is $V_n = 0.4356$, and the value coefficient when implementing the maintenance strategy is $V_s = 3.0429$. It can be seen that the value coefficient V_s after adopting the maintenance strategy is much higher than that without the maintenance strategy V_n . This shows that the maintenance strategy proposed in this study can effectively control the wear and fatigue of key components, slow down the speed of performance degradation, and significantly improve the charging square camera after its scheduled service life expires; the value coefficient of continuing service in the electromechanical system and its overall maintenance cost is kept at a relatively low level.

5. Recommended Configuration of Parameters with a High Failure Rate

From the fault prediction analysis results of the video surveillance equipment (Figure 9), we can see that even if the appropriate maintenance strategy is adopted, the probability of slight failure of the video surveillance equipment in the steady state is still as high as 9.61%, and the probability of general failure is 4.93%. This also intuitively shows that the degree of performance protection by the external maintenance of the mechanical and electrical equipment is limited, and it cannot guarantee that the equipment is at a healthy level at all times. There are still some equipment failures due to the impact of the external environment and their aging. If the reliability of electromechanical equipment can be further improved, it is necessary to improve the level of technical parameters of the equipment from the inside to improve its essential performance. Based on the above situation, this section still takes video surveillance equipment (mainly cameras) as an example, combined with its minor faults and general failures, to recommend the configuration of parameters to achieve the goal of fundamentally improving the quality of the equipment.

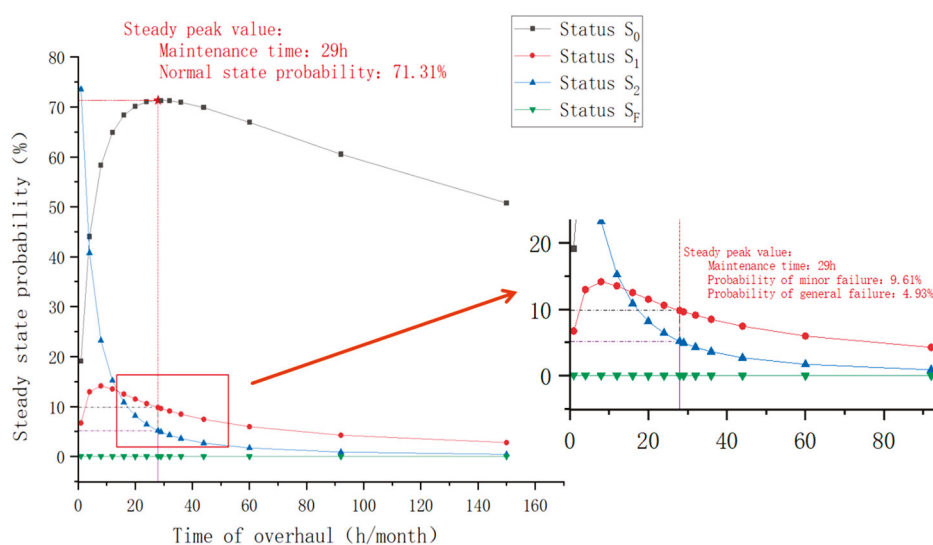


Figure 9. Probability change between the minor failure and general failure state of video surveillance equipment.

5.1. Carding of Technical Specifications

Video surveillance equipment must monitor the vehicle's driving situation and personnel's activities in the square after payment and provide clear video images in an emergency. Various cameras and video surveillance equipment have different requirements regarding visual resolution, working angle, signal processing mode [27], and reliability according to their distribution position. This project mainly aims at the relevant parameters affecting all kinds of cameras and compiles some general technical specifications of cameras in Chinese national standards GB/T 36480 2018 and GB/T 15412-2017 to provide a basis for setting the technical parameters of different cameras (Table 6).

Table 6. Technical specification for video surveillance equipment (camera) comparison table.

Related Parameters	Chinese National Standard: GB/T 36480~2018; GB/T 15412-2017
Total number of optically effective pixels	The optical effective pixel should not be less than 90% of the nominal value specified in the product standard.

Table 6. Cont.

Related Parameters	Chinese National Standard: GB/T 36480~2018; GB/T 15412-2017
Visual resolution	1. Products with a total number of nominal pixels less than 20 million pixels should not be less than 65% of the theoretical limit resolution; 2. Products with a total number of nominal pixels greater than or equal to 20 million pixels should not be less than 60% of the theoretical limit resolution.
Operating temperature range	−10 °C~55 °C
Working humidity range	30~85%
Geometric distortion	1. For products with more than 2 million nominal total pixels, the absolute value of pillow or barrel geometric distortion around the captured image is less than 3%; 2. The difference between the absolute value and the nominal value of geometric distortion is less than 5%.
Reliability parameters	1. After working continuously for 1000 h, the illuminance value of the lighting device should not be less than 90% of the initial illumination value; 2. The average time between failures (MTBF) of the head should not be less than 5000 h.
Head rotation angle	1. If the product stipulates that the horizontal (or vertical) rotation angle of the head is less than 360°, then under the condition of GB/T 15412.5.5.1, first rotate the head left (upward) to the limit position in the horizontal (or vertical) direction, and take this position as the starting point, and then rotate the head in the opposite direction to the limit position, and take this position as the terminating end, and the rotation angle between the starting end and the ending end is the rotation angle range. 2. If the product stipulates that the horizontal (or re-straightening) rotation angle of the head is greater than 360°, then under the condition of GB/T 15412.5.5.1, take a rest position of the head as the starting point, and then rotate the head horizontally (or vertically). If the head rotates to 360°, it can still return to the beginning; that is, the rotation angle range of the head is 360°.

5.2. Recommendation of Parameter Configuration of Video Surveillance Equipment

The slight and general failure performance analysis and parameter recommendations of all kinds of cameras for video surveillance equipment in the operating environment are as follows.

Under high temperatures, the lens of the toll plaza camera may be deformed due to thermal expansion and contraction, which will accelerate the degradation process of the internal electronic components, lead to image distortion, and then reduce its imaging quality and stability. Therefore, this kind of camera should have good high-temperature resistance. In northern Xinjiang, the low temperature in winter has a significant impact on the equipment. If moisture is inside the camera shield, the icing phenomenon may damage the equipment. In addition, the electronic components inside the equipment may be unstable at low temperatures, resulting in a decline in the performance of the equipment and maybe even causing difficulties in starting or sudden shutdown. Based on this, it is recommended that the working temperature and humidity supported by the camera be between −40 °C and 70 °C, and there should be no condensation when the humidity is less than 95%. Windblown sand particles in Xinjiang may be attached to the surface of the HD camera lens, resulting in visual field obstruction, which will reduce the image's resolution. At the same time, the wind sand may also cause wear and tear on the coating and lens, affecting its optical performance. Therefore, it is recommended that

the protection level of the outdoor camera is not lower than that of IP66. According to the above general specifications for outdoor cameras, the main technical parameters of cameras in different locations can be obtained. The following recommended configurations for toll plaza cameras, surveillance head cameras, and surveillance HD cameras are shown in Table 7.

Table 7. Recommended configuration table for video surveillance equipment (camera) parameters.

Parameters related to external environment	Anti-corrosion and anti-impact	(1) Toll plaza camera, surveillance head camera: 1. Corrosion resistance, impact resistance; 2. The column is treated by hot-dip galvanizing and then treated by plastic spray.
	Temperature and humidity range	(1) Toll Plaza camera: 1. Humidity less than 95% (no condensation); 2. Working temperature and humidity: $-40^{\circ}\text{C} \sim 70^{\circ}\text{C}$.
	Waterproof and dustproof	(1) Toll Square camera (gun camera): 1. It has the functions of dust prevention, fog prevention, and defrosting; 2. Protection level IP67; (2) Monitoring head camera: 1. Protection level: IP67, dustproof, antifogging, defrosting; 2. Installation of integrated aluminum outdoor protective cover, die-cast aluminum plate, stainless steel hardware, spray protection, waterproof seal; (3) Monitoring high-definition fixed cameras: 1. The protection level is not lower than that of IP66.
	Sight distance	(1) Charge plaza camera (gun camera), surveillance head camera: 1. Laser night vision distance: $\geq 500\text{ m}$; (2) Monitoring head camera: 1. The supplementary light distance is not less than 50 m.
Other performance parameters		(1) Toll square camera (gun camera): 1. Horizontal 360° infinite rotation, no dead angle; automatic cruise can be set, horizontal scanning; 2. Camera requires effective pixels: ≥ 4 million pixels; 3. Minimum illuminance: color $\leq 0.0003\text{ lx}$, black and white $\leq 0.0001\text{ lx}$; signal-to-noise ratio: $\geq 50\text{ dB}$; illuminance adaptation range: $\geq 120\text{ dB}$; 4. Lens near focal length: $\leq 6\text{ mm}$; far focal length: $\geq 208\text{ mm}$; 5. Lens image resolution: $\geq 2560 \times 1440$; 6. The head supports horizontal hand control speed not less than $200^{\circ}/\text{s}$ and vertical hand control speed not less than $83^{\circ}/\text{s}$. The horizontal rotation range is 360° continuous rotation, the vertical rotation range is upward $\geq 40^{\circ}$, downward $\geq 90^{\circ}$, and the positioning accuracy of the head is 0.01° ; 7. Mean time between failures (MTBF): $\geq 30,000\text{ h}$; (2) Monitoring head camera: 1. Effective pixels: ≥ 4 million pixels 2. Minimum illuminance: color $\leq 0.0003\text{ lx}$, black and white $\leq 0.0001\text{ lx}$; signal-to-noise ratio: $\geq 50\text{ dB}$; illuminance adaptation range: $\geq 120\text{ dB}$; 3. Near focal length: $\leq 6\text{ mm}$; far focal length: $\geq 208\text{ mm}$; 4. The angle of view of the lens is $59.8^{\circ}(\text{W}) \sim 2.0^{\circ}(\text{T})$, the horizontal manual speed is not less than $200^{\circ}/\text{s}$, and the vertical manual speed is not less than $83^{\circ}/\text{s}$. The horizontal rotation range is 360° continuous rotation, and the vertical rotation range is upward $\geq 40^{\circ}$ and downward $\geq 90^{\circ}$.

The above process reflects the strategies and steps for configuring technical parameters of video surveillance equipment, and this strategy can also be used to optimize the technical parameters of other equipment with high failure rates.

6. Conclusions

To manage expressway mechanical and electrical equipment, we should formulate the optimal external operation and maintenance strategy and improve the technical parameter level of the mechanical and electrical equipment inside the equipment. Based on the perspective of the operation and maintenance capability of electromechanical equipment, and integrating the historical operation and maintenance data, this study constructs a multistate electromechanical system Markov chain model considering maintenance, which aims to predict and analyze all kinds of fault types of the equipment. This study conducts an in-depth exploration of the equipment's tail operation and maintenance strategy, and based on the predicted results, optimizes the electromechanical equipment parameters for the fault performance of the equipment. The conclusions of the study are as follows:

(1) Taking video surveillance as an example, the fault maintenance data of two road sections in Xinjiang, China, in October 2023 are used to predict all kinds of faults of the electromechanical system as a whole, and the effectiveness of the prediction model is verified. Secondly, taking video surveillance equipment as an example, the failure rate of video surveillance equipment in different states is predicted, and an optimal maintenance strategy with a monthly maintenance time of 29 h and a maintenance frequency of 0.1648 is proposed. The overall expected operation probability of the long-term operation of the video surveillance equipment is increased to 71.31%, and the value coefficient of the video surveillance equipment under this strategy is verified.

(2) Based on the high incidence of minor and general faults in the predicted results, combined with the national and industry norms corresponding to the equipment, the equipment is configured with technical parameters suitable for the region to which it belongs. Improving its reliability from within the equipment is required to achieve this.

This study creatively combines the equipment's external operation and maintenance strategy with improving internal parameters. It takes video surveillance equipment as an example to explore the optimal operation and maintenance strategy and parameter configuration recommendation scheme for expressway mechanical and electrical equipment suitable for Xinjiang. The formed equipment operation and maintenance method can provide strong theoretical support and practical guidance for expressway mechanical and electrical equipment operation and maintenance method innovation and equipment selection in practical work. The current model constructed in this study only preliminarily classifies and analyzes the state of the equipment, but the state can be divided further; in the follow-up research, the study will include more states into the multistate Markov model of the current maintenance decision in order to achieve a finer division of the equipment state. At the same time, it is planned to optimize the relevant equipment parameters of the electromechanical system in order to promote the in-depth study of the operation and maintenance strategy of electromechanical equipment and provide clearer guidance for the optimization of electromechanical equipment parameters.

Author Contributions: Conceptualization, X.D.; Methodology, X.D. and G.S.; Software, G.S.; Formal analysis, G.S.; Data curation, X.D., G.S., W.T. and L.C.; Writing—original draft, X.D., G.S. and W.T.; Writing—review & editing, X.D., G.S. and L.C.; Project administration, X.D., W.T. and L.C. All authors have read and agreed to the published version of the manuscript.

Funding: This study is jointly funded by the Natural Science Foundation of Xinjiang Uygur Autonomous region (2024D01A53), Science and Technology Projects of Xinjiang Communications Investment Group Co., Ltd. (XJITZKX-FWCG-202401-0044), Xinjiang Key R&D Program Projects (2022B03033-1), and the project of Tianchi in Xinjiang Uygur Autonomous Region.

Data Availability Statement: The data presented in this study are available on request from the corresponding author due to privacy.

Conflicts of Interest: The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper. Wei Tian is employee of Xinjiang Jiaotou Engineering Technology Development Co., Ltd. and Xinjiang Transportation Investment (Group) Co., Ltd. The authors declare that this study received funding from Xinjiang Communications Investment Group Co., Ltd. The funder had no role in the design of the study; in the collection, analysis, or interpretation of data, in the writing of the manuscript, or in the decision to publish the results.

References

- Kim, D.J.; Sur, J.M.; Cho, H.U. A long term expected risk estimation of maritime accidents through markov chain approach and probabilistic risk matrix. *Asian J. Shipp. Logist.* **2023**, *39*, 1–12. [CrossRef]
- Li, D.Q.; Zang, H.H.; Tang, X.S.; Rong, G. Efficient Bayesian updating for deformation prediction of high rock slopes induced by excavation with monitoring data. *Eng. Geol.* **2024**, *342*, 107772. [CrossRef]
- Jia, X.; Papadimitriou, C. Data features-based bayesian learning for time-domain model updating and robust predictions in structural dynamics. *Mech. Syst. Signal Process.* **2025**, *224*, 112197. [CrossRef]
- Zhu, H.; Hao, H.K.; Lu, C. Enhanced support vector machine-based moving regression strategy for response prediction and reliability estimation of complex structure. *Aerosp. Sci. Technol.* **2024**, *155*, 109634. [CrossRef]
- Wang, Q.; Chen, D.; Li, M.; Li, S.; Wang, F.; Yang, Z.; Zhang, W.; Chen, S.; Yao, D. A novel method for petroleum and natural gas resource potential evaluation and prediction by support vector machines (SVM). *Appl. Energy* **2023**, *351*, 121836. [CrossRef]
- Zhao, J.; Zhang, H.; Zou, H.; Pan, J.; Zeng, C.; Xiao, S.; Wang, J. Probability prediction method of transmission line icing fault based on adaptive relevance vector machine. *Energy Rep.* **2022**, *8*, 1568–1577. [CrossRef]
- Hocek, H.; Yay, S.; Yazir, D. Comprehensive analysis of ship detention probabilities using binary logistic regression method with machine learning. *Ocean. Eng.* **2025**, *315*, 119889. [CrossRef]
- Li, C.; Xiong, S.; Xiong, H.; Sun, X.; Qin, Y. Logistic model for pattern inference of subway passenger flows based on fare collection and vehicle location data. *Appl. Math. Model.* **2024**, *130*, 472–495. [CrossRef]
- Huang, W.; Shao, C.; Hu, B.; Li, W.; Sun, Y.; Xie, K.; Zio, E.; Li, W. A restoration-clustering-decomposition learning framework for aging-related failure rate estimation of distribution transformers. *Reliab. Eng. Syst. Saf.* **2023**, *232*, 109043.
- Fan, L.; Xu, X.; Ni, Y. Multi-failure mode reliability of monorail vehicle gear transmission system based on multi-index staged degradation. *Appl. Math. Model.* **2024**, *136*, 115602.
- Yuan, X.; Xu, M.; Hou, Z.; Chen, W.; Huang, Y.; Lv, J.; Xu, X.; Huang, L. Enhanced data-driven economic assessment of fuel cell electric buses utilizing an improved markov chain monte carlo approach. *Int. J. Hydrogen Energy* **2025**, *102*, 732–748.
- Artan, M.Ş.; Şahin, İ. A comparative analysis of train delay prediction models for markov chains. *Transp. Res. Procedia* **2025**, *82*, 822–835.
- Wang, X.; Liu, W. Research on air traffic control automatic system software reliability based on markov chain. *Phys. Procedia* **2012**, *24*, 1601–1606.
- Yu, Q.; Guo, Z.Z.; Liang, R. A prediction model of expressway electromechanical equipment life based on markov chain. *J. Highw. Transp. Res. Dev.* **2024**, *35*, 28–35.
- Zhu, D.; Ding, Z.; Huang, X.; Li, X. Probabilistic modeling for long-term fatigue reliability of wind turbines based on Markov model and subset simulation. *Int. J. Fatigue* **2023**, *173*, 107685.
- Liang, Q.; Yang, Y.; Zhang, H.; Peng, C.; Lu, J. Analysis of simplification in Markov state-based models for reliability assessment of complex safety systems. *Reliab. Eng. Syst. Safety* **2022**, *221*, 108373. [CrossRef]
- Haghgoo, O.; Damchi, Y. Reliability modelling of capacitor voltage transformer using proposed Markov model. *Electr. Power Syst. Res.* **2022**, *202*, 107573. [CrossRef]
- Yi, Y.; Siyu, H.; Haoran, C.; Meilin, W.; Linhan, G.; Xiao, C.; Liu, W. Belief reliability analysis of traffic network: An uncertain percolation Semi-Markov model. *J. Frankl. Inst.* **2023**, *360*, 12463–12479. [CrossRef]
- Zhang, X.; Li, Y.; Zhao, Z.; Zhang, J.; Zhang, W. Reliability assessment of multistate flexible manufacturing cells considering equipment failures. *Comput. Ind. Eng.* **2023**, *185*, 109625. [CrossRef]

20. Sun, C.; Ju, X.; Hao, W.; Lu, Y. Research on multi-objective optimization of control strategies and equipment parameters for a combined heating system of geothermal and solar energy in cold and arid regions based on TRNSYS. *Case Stud. Therm. Eng.* **2023**, *50*, 103441. [CrossRef]
21. Li, X.Y.; Wu, J.P.; Ma, H.G.; Li, X.; Kang, R. A random fuzzy accelerated degradation model and statistical analysis. *IEEE Trans. Fuzzy Syst.* **2017**, *26*, 1638–1650.
22. Li, N.; Lei, Y.; Guo, L.; Yan, T.; Lin, J. Remaining useful life prediction based on a general expression of stochastic process models. *Trans. Ind. Electron.* **2017**, *64*, 5709–5718.
23. Zarei, M.; Bagheri, M.; Dehghanian, P. Markov-chain-driven optimization of inspection-based maintenance, part I: Models and methods. *Electr. Power Syst. Res.* **2024**, *228*, 110049.
24. Zarei, M.; Bagheri, M.; Dehghanian, P. Markov-chain-driven optimization of inspection-based maintenance, part II: Numerical analysis and practical insights. *Electr. Power Syst. Res.* **2024**, *227*, 109922.
25. Yan, L. Application of value engineering to project construction cost control by the management engineers. *J. Northeast. For. Univ. Chin. Ed.* **2002**, *30*, 101–103.
26. Mabrouka, S.; Younis, E. Application of value engineering technique in construction project. *J. Eng. Appl. Sci. Technol.* **2023**, *5*, 1–6. [CrossRef]
27. Panic, S.; Petrović, V.; Drašković, S.; Kontrec, N.; Milojević, S. Performance analysis of hybrid fso/rf communication system with receive diversity in the presence of chi-square/gamma turbulence and rician fading. *Bull. D. Serikbayev EKTU* **2023**, *4*, 304–312.

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Research on Construction Countermeasures for Freeze–Thaw Deformation of Permafrost Subgrade in Forest Regions of Northeast China

Zhibo Xu ¹, Guihe Wang ^{2,*} and Zhu Yang ³

¹ School of Civil Engineering and Transportation, Beihua University, Jilin 132013, China; xuzhibo@beihua.edu.cn

² School of Engineering and Technology, China University of Geosciences (Beijing), Beijing 100083, China

³ School of Forestry, Beihua University, Jilin 132013, China; yz1990@beihua.edu.cn

* Correspondence: wanggh@cugb.edu.cn

Abstract: The permafrost in forest regions of Northeast China is very sensitive to the disturbance of subgrade construction, which will aggravate the degradation of the permafrost upper limit, leading to freeze–thaw deformation of the permafrost subgrade. Based on the road construction project of Highway 332 in forest regions in Northeast China, through theoretical analysis, indoor experiments, on-site monitoring, and numerical simulation methods, a thermo-hydro-mechanical coupling numerical model of the permafrost subgrade was established. A “two-step” construction countermeasure for freeze–thaw deformation of permafrost subgrade based on rubble stone subgrade structure was proposed. The study indicates that the addition of rubble stones to the subgrade structure has a significant cooling effect. The optimal thickness for filling rubble stones is 1 m. The optimal construction timing for subgrade is a two-step construction across the year. The stamping construction of the rubble stones is in November. The filling construction of rubble stones and gravel is in April and May of the next year. Based on the proposed construction countermeasure for permafrost subgrade, the settlement at the center of the subgrade surface is 12.7 mm in the 5th year, 17.6 mm in the 10th year, 21.1 mm in the 15th year, and 23.5 mm in the 20th year. The settlement deformation of the subgrade tends to stabilize, which can ensure the long-term stability and safety of road operations.

Keywords: construction countermeasure; permafrost subgrade; freeze–thaw deformation; forest region

1. Introduction

Permafrost is widely distributed in the Greater Khingan Mountains in Northeast China, which is characterized by thin overburden, shallow natural upper limit, and high volume ice content [1,2]. The frozen soil is easier to be disturbed by the construction of subgrade, leading to the degradation of the permafrost upper limit and the freeze–thaw deformation of the subgrade [3]. After the field survey, it was found that the permafrost roads indeed suffered serious frost damage [4,5]. Therefore, the conventional subgrade construction countermeasures cannot effectively reduce the freeze–thaw deformation of the subgrade. It is necessary to put forward construction countermeasures that can alleviate the deformation of the subgrade in forest regions.

At present, the protective measures for permafrost subgrade widely used in engineering construction mainly include the following: ventilation pipe, heat rod, insulation

board, and rubble stone [6–11]. Yuan et al. [12] took the hollow block ventilation subgrade as the research object and found that the hollow block ventilation structure can effectively take the heat inside the subgrade and reduce the temperature of the subgrade. Cheng et al. [13] found that the composite ventilated subgrade had a good hypothermal effect. However, during high-temperature periods, the function of the heat exchange of the ventilation pipe will increase the heat inside the roadbed [14]. Li et al. [15] found that the hot rod can effectively improve the upper limit of the frozen soil under the Qinghai–Tibet railway subgrade, and the effect is better when the hot rod is inserted obliquely. Liu et al. [16] studied the effective influence radius, duration, and depth of L-shaped and straight heat rods, as well as their cooling effect on frozen soil subgrade. The cooling effect produced by the hot rod can only affect part of the subgrade area near the hot rod. Tai et al. [17] analyzed the degradation law of the frozen soil upper limit of insulation board subgrade under various working conditions. Zhang et al. [18] found that the insulation board has a protective effect on permafrost. However, the function of the insulation board is conditional. The insulation board can only play its role in the warm season. At this time, the internal temperature of the permafrost is lower than the external temperature. Chen et al. [19] found that the rubble stone subgrade can effectively eliminate the melting interlayer along the Qinghai–Tibet highway. At the same depth, the temperature of ordinary subgrade is relatively higher than that of rubble stone subgrade. The rubble stones can effectively protect the underlying frozen soil. Ma et al. [20] analyzed and verified the effectiveness of rubble stone subgrade structure in improving subgrade settlement from the perspective of subgrade thermal stability.

Based on the interlocking effect of rubble stones in the roadbed, it can improve the bearing capacity of the roadbed and prevent capillary water from rising into the subgrade. Rubble stones can also remove the heat inside the subgrade through convection under low-temperature conditions, and absorb less heat under high-temperature conditions. The underlying soil of the roadbed can be effectively cooled [21,22]. The long winter and low temperature in Northeast China also contribute to the heat dissipation of rubble stone subgrade. In the aspect of economy, the cost of rubble stones is relatively low, especially in areas with abundant stone resources. Local stones can be used, greatly reducing the procurement and transportation costs of rubble stones. In addition, the construction process of rubble stones is relatively simple, and the technical requirements for construction personnel are not particularly high, resulting in relatively low labor costs. It can be seen that the rubble stones are more suitable for permafrost subgrade. In summary, the application objects of the subgrade protection measures studied by scholars are mostly the road engineering of the Qinghai–Tibet Highway and Qinghai–Tibet Railway. In the Northeast region, the natural conditions, permafrost characteristics, and vegetation coverage are different from those in the Qinghai–Tibet region. The rubble stone subgrade is mostly used in railway engineering, and is rarely used for high-grade roads in highway engineering. There is even less research on the rubble stone subgrade in permafrost forest regions in Northeast China.

Therefore, based on the road construction project of Highway 332 (referred to as G332) in forest regions in Northeast China, rubble stone was added to the subgrade structure to improve the stability of the subgrade and a “two-step” construction countermeasure for freeze–thaw deformation of permafrost subgrade, based on rubble stone subgrade structure, was proposed. It can alleviate the disturbance caused by subgrade construction to underground permafrost and help to maintain the long-term safety and stability of permafrost subgrade construction and operation in forest regions.

2. Materials and Methods

2.1. Study Area

Figure 1 shows that the road construction project of G332 in this study is situated in the permafrost region of the Great Khingan Mountains in Northeast China, connecting Oroqen Autonomous Banner and Genhe City. The distance of the construction road in the forest region is 117 km. The design speed of the road is 80 km/h, and the design width of the subgrade is 12 m. As shown by the red mark in Figure 1, it is the K104+000 subgrade section researched in this study. Figure 2 shows the actual scene of the K104+000 subgrade section.



Figure 1. Location of the construction section of G332 in Northeast China.



Figure 2. Actual scene of the K104+000 subgrade section.

In the study area, the annual temperature difference is large, and the temperature changes dramatically [23]. Summer is short, and precipitation is concentrated. The winter is long and cold. The surface water system here is developed, and the groundwater is rich. Based on geological surveys conducted on site, the soil here is wet and soft. The surface layer is covered with thick humus soil. The underground is quaternary alluvial, proluvial clay, pebble, and volcanic breccia. The upper limit of frozen soil is -1.5 m, and the lower limit is -12 m. There are a large number of ice lenses.

The vegetation along the road is dense with a coverage rate of 83%, and the vegetation types are mainly *Larix gmelinii* and *Betula platyphylla*. The dense vegetation cover provides natural conditions for the preservation of permafrost. Permafrost and vegetation coexist and constrain each other. Vegetation can reduce the heat radiation of the sun to the ground and the thermal energy conducted to the underground frozen soil layer. It is helpful to reduce the degradation rate of frozen soil [24–28].

2.2. Field Observation Methods

As shown in Figure 3, PT100 thermocouple temperature sensors, RS458 solid moisture sensors, and digital level were used to monitor the temperature, volumetric moisture content, and settlement of the K104+000 permafrost subgrade section [29–34]. T1 and T2 were boreholes for embedding temperature sensors. W1 and W2 were cross-sections for placing moisture sensors. The monitoring points for subgrade temperature and volumetric moisture content were placed outside the slope toe of the subgrade. The monitoring marks for subgrade settlement were evenly distributed on the surface of the subgrade. The temperature, volumetric moisture content, and settlement monitoring began in October 2019, among which the settlement monitoring in the center of the subgrade began after the completion of construction in May 2020.

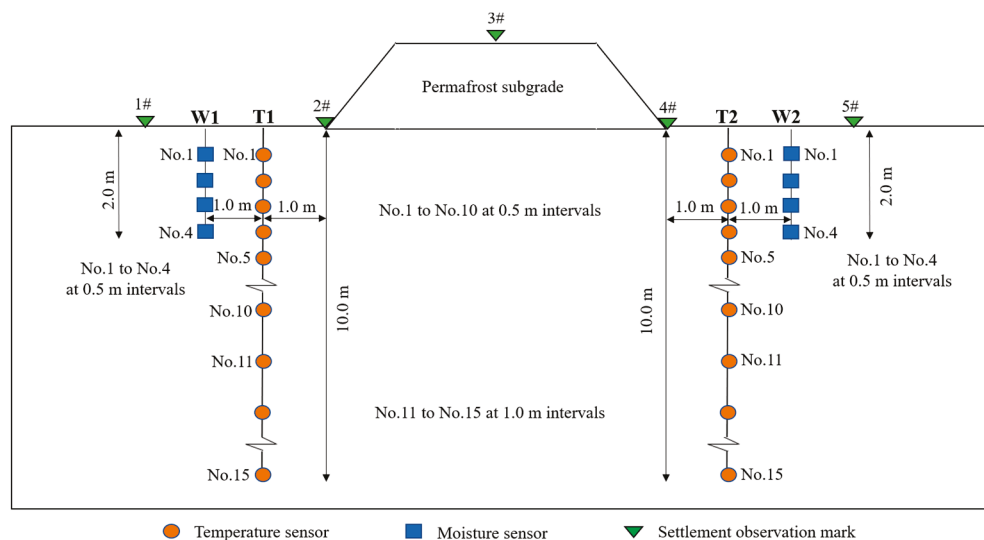


Figure 3. The distribution of monitoring points for K104+000 permafrost subgrade.

2.3. Establishment of a Numerical Model for Permafrost Subgrade

In this study, the numerical model is based on the following assumptions: (1) the frozen soil is uniformly continuous and isotropic; (2) in the study of water field, it is assumed that the migration law of water follows the generalized Darcy’s law; (3) the water migration between unfrozen soil and frozen soil occurs in liquid form, ignoring gas-phase migration; and (4) the deformation of soil skeleton caused by temperature changes is not considered. The phase transition of ice water and the migration of water in unsaturated frozen soil were considered. Based on the thermal conduction, moisture migration, and stress–strain field control equation of permafrost, a thermo-hydro-mechanical coupling theoretical model for frozen soil in forest regions was established. Subsequently, the theoretical model was introduced into the partial differential equation (PDE) module of COMSOL Multiphysics 5.6 finite element software for secondary development. Finally, a numerical model of thermo-hydro-mechanical coupling for permafrost subgrade in forest regions was established [35–37].

Based on the permafrost subgrade numerical model, a two-dimensional finite element model of the subgrade at the K104+000 section was established in COMSOL Multiphysics 5.6. As shown in Figure 4, the model is 40 m wide and 10 m high. The subgrade is 12 m wide and 3 m high. The subgrade filling is composed of gravel and rubble stones. According to geological survey data, the soil is relatively wet and soft. The underlying soil of the subgrade is composed of clay, round gravel, and fully weathered volcanic breccia. Therefore, in the model, soil from 0 to 2.5 m underground is clay, soil from 2.5 m to 4.8 m is a round gravel layer, and soil from 4.8 m to 10 m is weathered volcanic breccia.

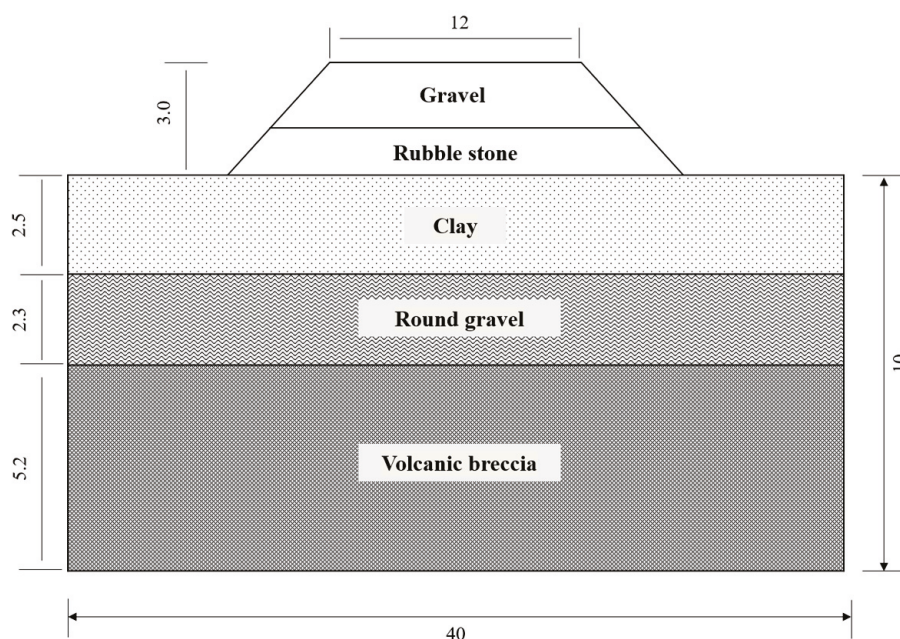


Figure 4. The schematic diagram of the K104+000 subgrade model (unit: m).

In the subgrade model, the upper boundary condition was based on the boundary layer principle. The temperature monitoring data, which was 0.5 m below the surface, was used to calculate the temperature field. Both the left and right boundaries were thermal insulation. Based on the on-site temperature monitoring data, the lower boundary temperature was constant with a value of $-1\text{ }^{\circ}\text{C}$. The moisture boundary conditions on both sides and the bottom were assumed to be impermeable, and the upper boundary was open. For displacement boundary conditions, only the upper boundary is free.

2.4. Material Parameters

The parameters of subgrade materials in the model were obtained through field and laboratory experiments, including the experiment of density, uniaxial compression, thermal conductivity, and specific heat capacity [38,39]. To simplify the calculation, the elastic modulus, thermal conductivity, and specific heat capacity were only considered in the freeze–thaw state in the model.

The density of the materials was determined by the ring knife method. The experimental instruments were a ring knife and a balance. The inner diameter of the ring knife is 50.46 mm and the height is 50 mm. The density of the materials is shown in Table 1.

Table 1. Physical parameters of the model.

Material	ρ_d (kg/m ³)	μ	E_u (Mpa)	E_f (Mpa)
Gravel	2000	0.30	78	110
Rubble stone	2400	0.30	150	180
Clay	1700	0.23	2.05	58
Round gravel	2100	0.25	107	140
Volcanic breccia	1950	0.24	53	98

Note: E_u is the elastic modulus of unfrozen soil; E_f is the elastic modulus of frozen soil.

The elastic modulus of the materials was determined through uniaxial compression experiments. The saturated material was placed in a fully automatic freeze–thaw testing machine for freeze–thaw testing. The freezing temperature was set as $-25\text{ }^{\circ}\text{C}$ and the melting temperature was set as $25\text{ }^{\circ}\text{C}$. Frozen and melted materials were subjected to uniaxial compression tests and loaded until material failure occurred. The elastic modulus of the materials under freeze–thaw conditions was measured. The elastic modulus of the materials is shown in Table 1.

The thermal conductivity of the materials was determined by the transient plane heat source method. The DRE-III multifunctional rapid thermal conductivity tester was used. This instrument used a Hot Disk probe, which can directly measure heat propagation without being affected by contact thermal resistance. The thermal conductivity of the materials in the freeze–thaw state was measured at $-25\text{ }^{\circ}\text{C}$ and $25\text{ }^{\circ}\text{C}$, respectively. The thermal conductivity of the materials is shown in Table 2.

Table 2. Thermal parameters of the model.

Material	λ_u (W/(m $\cdot^{\circ}\text{C}$))	λ_f (W/(m $\cdot^{\circ}\text{C}$))	C_u (J/(kg $\cdot^{\circ}\text{C}$))	C_f (J/(kg $\cdot^{\circ}\text{C}$))
Gravel	1.96	2.31	880	790
Rubble stone	1.04	2.84	820	740
Clay	1.32	1.83	1190	1080
Round gravel	1.56	1.97	1010	920
Volcanic breccia	1.78	2.19	940	840

Note: λ_u is the thermal conductivity of unfrozen soil; λ_f is the thermal conductivity of frozen soil; C_u is the specific heat of unfrozen soil; C_f is the specific heat of frozen soil.

The specific heat capacity of materials was determined by the cooling mixing method. The BRR specific heat capacity tester was used. This instrument adopted high-precision temperature-measuring thermocouples and temperature-measuring instruments, which can improve the accuracy of experiments. The specific heat capacity of the materials in the freeze–thaw state was measured at $-25\text{ }^{\circ}\text{C}$ and $25\text{ }^{\circ}\text{C}$, respectively. The specific heat capacity of the materials is shown in Table 2.

3. Results

3.1. Numerical Model Validation

After the establishment of the numerical model for the subgrade, the model can be solved. The time unit for numerical simulation is days, and the time step is 1 day.

Due to the fact that the numerical model established in this study was used to calculate the stability of permafrost subgrade in forest regions under long-term operation, the accuracy of the model needed to be fully validated. The model calculation results and on-site monitoring data within one year after the completion of subgrade construction (from May 2020 to May 2021) were selected for comparison. The on-site monitoring data have been obtained in Section 2.2. The period within one year after the completion of subgrade construction was selected for numerical simulation, with a calculation time of

12 months (360 days). The temperature, unfrozen water content, and settlement of permafrost subgrade were calculated.

Figure 5 shows the numerical simulation and on-site measured temperature data at a distance of 10 m on the left side of the subgrade center (borehole T1) from May 2020 to May 2021. The simulated value and the measured value are basically the same. Figure 6 shows the numerical simulation and on-site measured unfrozen water content data at a distance of 11 m on the left side of the subgrade center (profile W1) from May 2020 to May 2021. The simulated value and the measured value are basically the same. Figure 7 shows the numerical simulation and on-site measured settlement in the center of the subgrade surface from May 2020 to May 2021. The simulated value and the measured value are basically the same. In summary, it can be seen that the numerical simulation calculations of the temperature, unfrozen water content, and settlement of the subgrade are correct and reliable.

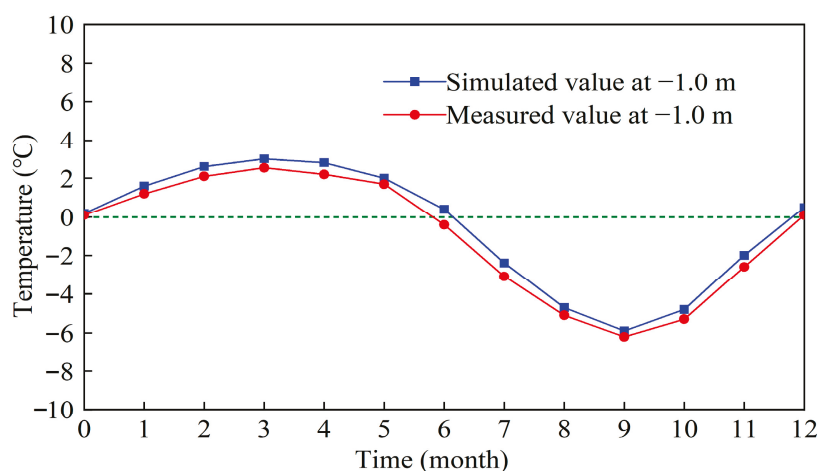


Figure 5. Comparison curve between simulated and measured temperature values at 1 m underground from May 2020 to May 2021.

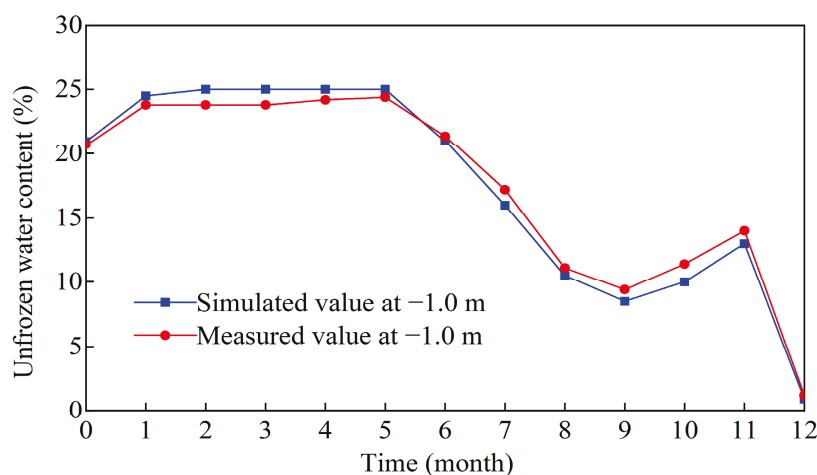


Figure 6. Comparison curve between simulated and measured underground unfrozen water content at 1 m underground from May 2020 to May 2021.

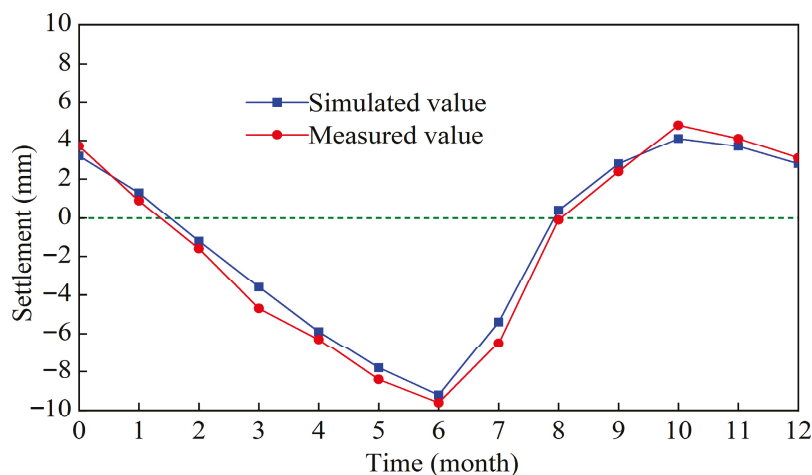


Figure 7. Comparison curve between simulated and measured settlement in the center of the subgrade surface from May 2020 to May 2021.

3.2. Analysis of the Optimal Permafrost Subgrade Structure

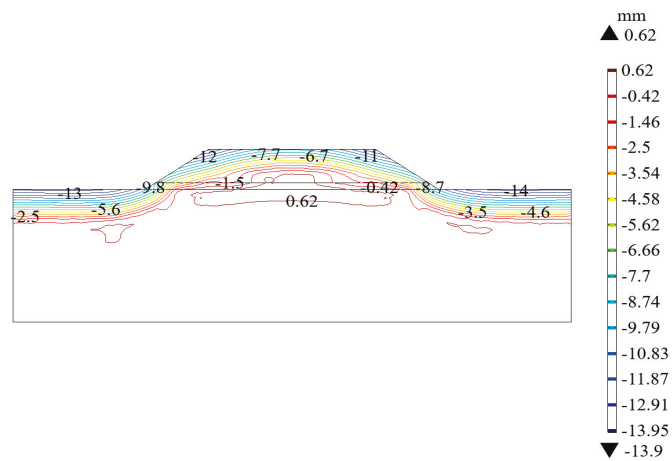
Considering the characteristics of permafrost in forest regions, rubble stones were added to the subgrade structure to optimize it. Firstly, to enhance the strength of the roadbed, block stones can be impacted and compacted below the surface. Its interlocking effect can reduce the compression deformation caused by subsequent subgrade construction. Meanwhile, it can hinder capillary water from rising into the subgrade structure [40–43]. The thickness of the impact compaction layer was determined by the geological conditions on site. The thickness here was 1 m. Then, the rubble stones were filled on the compacted layer, which can play a role in ventilation and heat dissipation [44,45]. However, there was no unified construction standard for the thickness of the rubble stone filling layer of permafrost subgrade in forest regions. Further discussion was needed to determine the optimal filling thickness. Finally, the gravel layer was filled. The filling thickness of the gravel layer depended on the design elevation.

The thickness of the rubble stone filling layer was set to 0.5 m, 1.0 m, and 1.5 m, respectively. The average particle size of the rubble stones was set to 20 cm, and the porosity was between 25 and 30%. Based on the established subgrade model, a numerical simulation was carried out. The time of significant subgrade settlement within one year (1 November) was selected for comparative analysis of the distribution patterns of subgrade deformation fields. Figure 8 shows the settlement distribution of the subgrade with three different rubble stone filling layers on 1 November. A positive settlement value represents the uplift of the subgrade, while a negative settlement value represents the subsidence of the subgrade. Figure 9 shows the settlement at the center, shoulder, and slope toe of the subgrade with three different rubble stone filling layers on 1 November.

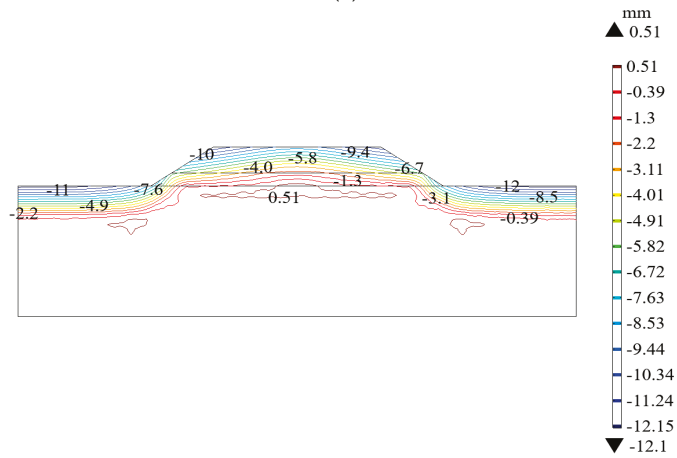
According to Figures 8 and 9, it can be seen that, at the center of the subgrade, the settlement of filling 0.5 m and 1 m rubble stone subgrades was significantly smaller than that of filling 1.5 m rubble stones. At the shoulder of the subgrade, the settlement of filling 1.5 m and 1 m rubble stones was significantly smaller than that of filling 0.5 m rubble stones. At the slope toe of the subgrade, the settlement of filling 1.5 m and 1 m rubble stones was also less than that of filling 0.5 m rubble stones. Therefore, considering the protective effect and construction cost, the optimal thickness for filling rubble stones was determined to be 1 m.

The optimal permafrost subgrade structure in the forest region was obtained. Firstly, a one-meter-thick layer of rubble stones was crushed and impacted below the surface. Then, a one-meter-thick layer of rubble stones was filled, with an average particle size of 20 cm

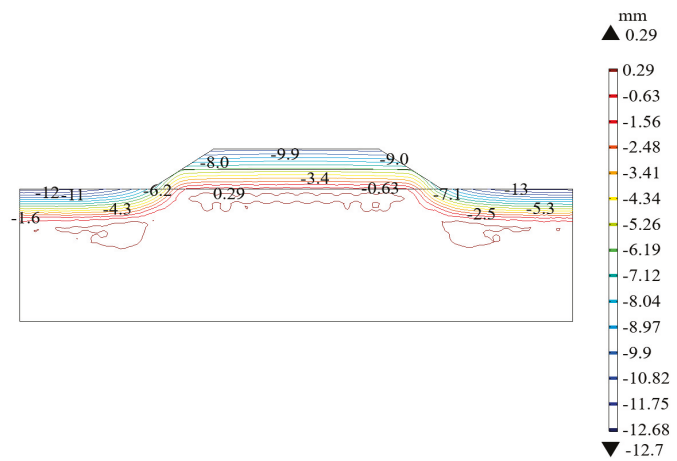
and a porosity between 25 and 30%. Finally, a two-meter-thick layer of gravel was filled. The total height of the subgrade is three meters.



(a)



(b)



(c)

Figure 8. Settlement distribution of subgrade with different thicknesses of rubble stone filling layers on November 1st: (a) 0.5 m; (b) 1.0 m; (c) 1.5 m.

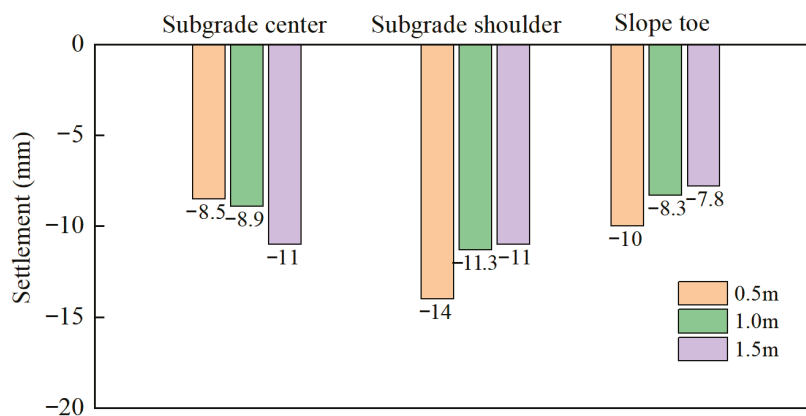


Figure 9. Comparison of settlement at different positions on the surface of subgrades with different thicknesses of rubble stone filling on November 1st.

3.3. Analysis of the Optimal Construction Timing for Subgrade

In order to minimize the construction damage to permafrost subgrade, the most suitable construction timing should be selected. Considering the experience of subgrade construction in permafrost regions and the actual construction conditions on site, it was determined that the optimized subgrade construction period would take approximately three or four months. At the same time, the winter in high-latitude forest regions is long, with extremely low temperatures and heavy snow blocking the mountains, making construction difficult from December to March each year. Therefore, continuous construction can be carried out from April to July. This type of construction timing was called continuous construction during the warm season.

In addition, considering the particularity of permafrost in forest regions, this study proposed a new two-step construction timing that spans years. The subgrade was cleared in October. The construction of rubble stone compaction was carried out before the freezing in November. No construction in winter. At that time, the temperatures were all negative, which can fully pre-cool the subgrade. In April of the next year, rubble stones were filled. The gravel was filled in May. This type of construction timing was called two-step construction across the year.

In this study, two types of construction timing, two-step construction across the year and continuous construction during the warm season, were simulated. The distribution pattern of subgrade settlement on November 1st of one year was analyzed. Their calculation results were compared to determine the optimal construction timing.

Figure 10 shows the settlement distribution of the subgrade after the two-step construction across the year and continuous construction during the warm season on November 1st. Figure 11 shows the settlement at different positions on the surface of the subgrades after the two-step construction across the year and continuous construction during the warm season on November 1st.

According to Figures 10 and 11, it can be seen that the surface of the subgrade was in a state of melting and sinking on November 1st. At the center of the subgrade surface, the settlement constructed in two steps across the year was 8.5 mm and the settlement constructed continuously in the warm season was 11.4 mm. At the shoulder of the subgrade, the settlement constructed in two steps across the year was 12.3 mm and the settlement constructed continuously in the warm season was 13.6 mm. At the slope toe of the subgrade, the settlement constructed in two steps across the year was 10.2 mm and the settlement constructed continuously in the warm season was 11.6 mm.

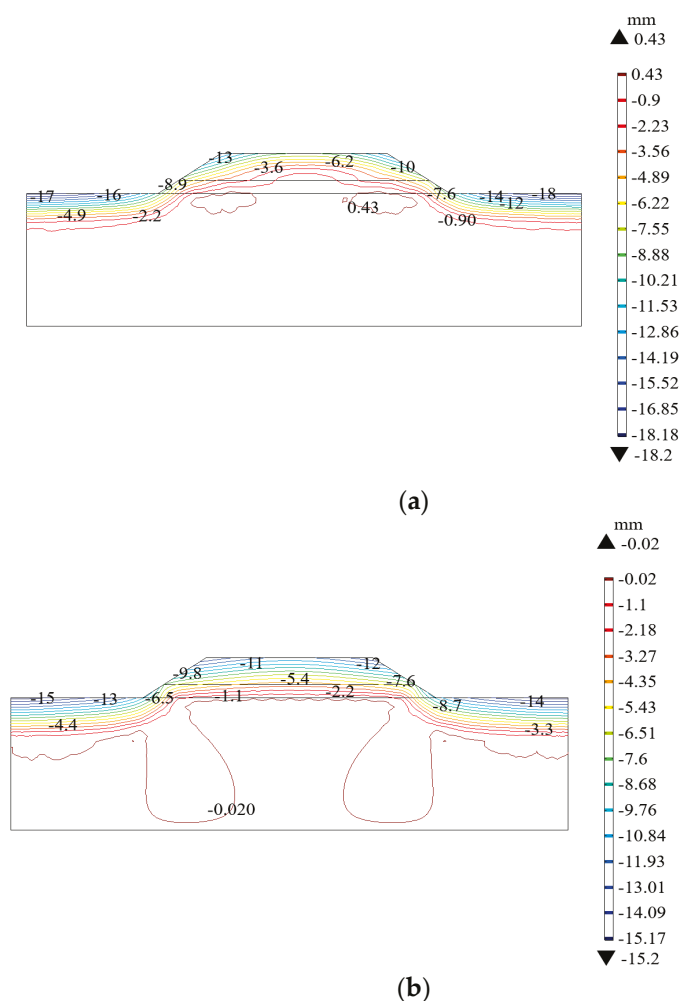


Figure 10. Settlement distribution of subgrade after two construction timings on November 1st: (a) two-step construction across the year; (b) continuous construction during the warm season.

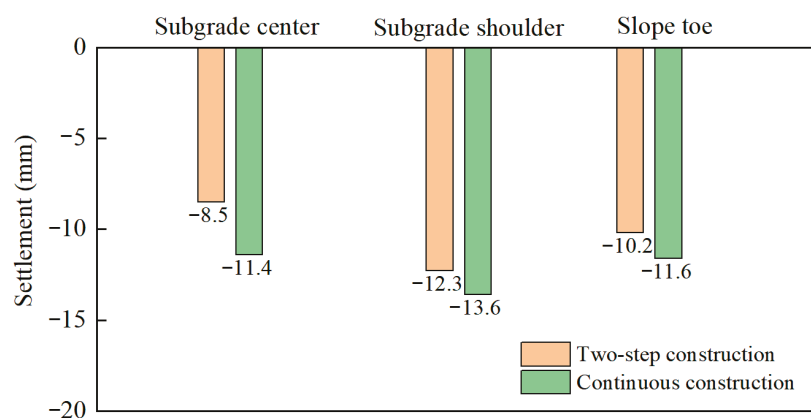


Figure 11. Comparison of settlement at different positions on the surface of subgrades between two construction timings on November 1st.

According to the data above, the settlement of the subgrade after the two-step construction across the year was relatively smaller than that after continuous construction during the warm season. It was because the temperature of the subgrade constructed in two steps across the year was relatively low, and the disturbance to the underlying permafrost was relatively small. Therefore, the settlement was relatively small in warm seasons. However, the temperature of the subgrade constructed continuously during the warm season was

relatively high, and the depth of permafrost melting was relatively large. Therefore, the settlement was relatively large in warm seasons, which caused serious damage to the road.

In summary, by comparing and analyzing two different construction timings, it was found that the two-step construction across the year was more conducive to the protection of frozen soil.

4. Discussion

After analyzing the optimal structure and construction timing of the permafrost subgrade in forest regions, a “two-step” construction countermeasure for freeze–thaw deformation of permafrost subgrade based on rubble stone subgrade structure was proposed. Firstly, the subgrade was cleared at 5 °C (in October). Secondly, a one-meter-thick layer of rubble stones was crushed and impacted below the surface at −4 °C (in November). Construction was stopped in winter. Thirdly, a one-meter-thick layer of rubble stones was filled when the temperature rose to 0 °C in the next year (in April). Finally, a two-meter-thick gravel layer was filled at 9 °C (in May).

The long-term applicability of the proposed construction countermeasure for permafrost subgrade should be discussed and evaluated. On November 1st of each year, within 20 years after the completion of the road, the variation law of settlement was discussed. Figure 12 shows the time-varying curve of settlement at the center of the subgrade surface on November 1st each year.

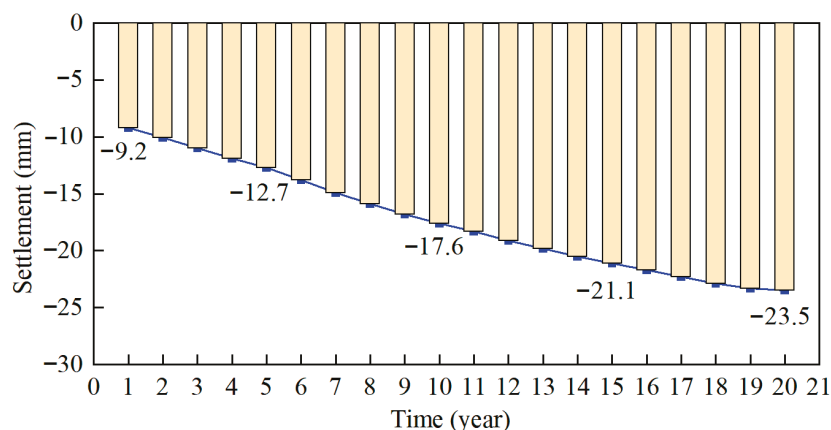


Figure 12. The time-varying curve of settlement at the center of the subgrade surface on November 1st each year.

According to Figure 12, in November each year, the external temperature decreased to negative values. However, the temperature inside the subgrade was still relatively high and the depth of permafrost melting was still increasing. The surface settlement of the subgrade was approaching its maximum value. The settlement at the center of the subgrade surface was 12.7 mm in the 5th year, 17.6 mm in the 10th year, 21.1 mm in the 15th year, and 23.5 mm in the 20th year.

The water–heat balance of permafrost was disrupted by subgrade construction, resulting in continuous accumulation of heat inside the subgrade. The depth of permafrost melting continued to increase and the permafrost upper limit continued to degrade. In the early stage of operation after construction, the rate of subgrade settlement was relatively fast. The stability of the subgrade was relatively poor. At this time, the permafrost beneath the subgrade was in a degraded state. As time went by, the disturbance of permafrost by external environmental changes gradually weakened. The rate of increase in subgrade settlement gradually slowed down. In the end, the water and heat changes in permafrost gradually reached a new equilibrium state, and the degradation amplitude of the per-

mafrost upper limit was relatively small. Afterwards, the settlement of the subgrade would remain stable for a long time. This indicated that the construction countermeasure for permafrost subgrade in forest regions was effective and feasible.

Two major limitations in this study could be addressed in future research. First, the stability of permafrost subgrade in forest regions is highly susceptible to climate change [46–49]. The model in this study is based on historical climate data and does not take into account the factor of climate change, which may lead to long-term prediction bias. Therefore, a remote long-term automatic monitoring system with automatic collection and transmission functions is planned to be developed. This system consists of a GPRS wireless communication system, solar power supply, and monitoring system. Among them, the monitoring system includes temperature sensors, moisture sensors, and layered settlement meters. Based on this system, remote monitoring and real-time collection of data from multiple cross-sections and measurement points can be achieved. Second, this study is based on the analysis of individual road sections, with a focus on the research of permafrost subgrades in forest regions. Moreover, only the construction countermeasures for filling the subgrade were discussed. In order to promote the research results more widely, various road sections, such as excavated subgrade, can be studied in the future.

5. Conclusions

Based on the road construction project of G332 in forest regions in Northeast China, through theoretical analysis, indoor experiments, on-site monitoring, and numerical simulation methods, a “two-step” construction countermeasure for freeze–thaw deformation of permafrost subgrade based on rubble stone subgrade structure was proposed. The conclusions are as follows:

- (1) The addition of rubble stones to the subgrade structure has a significant cooling effect. The optimal subgrade structure includes a one-meter-thick layer of rubble stone stamping, a one-meter-thick layer of rubble stone filling, and a two-meter-thick layer of gravel.
- (2) The settlement of the subgrade after the two-step construction across the year is relatively smaller than that after continuous construction during the warm season. The optimal construction timing for subgrade is a two-step construction across the year. The stamping construction of the rubble stones is in November. The filling construction of rubble stones and gravel is in April and May of the next year.
- (3) The settlement of the subgrade increases rapidly at the beginning of road operation; As time goes by, the rate of increase in settlement slows down significantly. Finally, the settlement of the subgrade gradually tends to stabilize. The proposed construction countermeasure for freeze–thaw deformation of permafrost subgrade can ensure the long-term stability and safety of road operations.

Author Contributions: Conceptualization, G.W.; formal analysis, Z.X. and Z.Y.; methodology, Z.X.; software, Z.X.; validation, Z.X.; investigation, Z.X. and Z.Y.; resources, G.W.; data curation, Z.X.; writing—original draft, Z.X. and Z.Y.; writing—review and editing, Z.X.; visualization, Z.Y.; supervision, G.W.; funding acquisition, Z.X. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the science and technology research project of the Jilin Provincial Department of Education. (NO. JJKH20240089KJ).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding author.

Acknowledgments: We would like to thank all staff members who contributed to this study who are not named here.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Gong, T.; Gao, B.; Ji, Z.; Cao, H. Variation of active layer thickness of permafrost in the Qinghai-Tibetan Plateau based on MODIS temperature product. *Sci. Geogr. Sin.* **2022**, *42*, 1848–1856. [CrossRef]
2. Nitzbon, J.; Westermann, S.; Langer, M.; Martin, L.C.P.; Strauss, J.; Laboor, S.; Boike, J. Fast response of cold ice-rich permafrost in northeast Siberia to a warming climate. *Nat. Commun.* **2020**, *11*, 2201. [CrossRef]
3. Grandpré, I.; Fortier, D.; Stephani, E. Degradation of permafrost beneath a road embankment enhanced by heat advected in groundwater. *Can. J. Earth Sci.* **2012**, *49*, 953–962. [CrossRef]
4. Yang, L.; Shang, Y.; Dai, C.; Liu, Y.; Li, G.; Gao, K.; Wu, Y.; Wei, Y. Evaluating the effectiveness of high-frequency ground-penetrating radar in identifying active layer thickness in the Da Xing'anling Mountains. *Remote Sens.* **2025**, *17*, 3484. [CrossRef]
5. Huang, Y.; Niu, F.; Chen, J.; He, P.; Yuan, K.; Su, W. Express highway embankment distress and occurring probability in permafrost regions on the Qinghai-Tibet Plateau. *Transp. Geotech.* **2023**, *42*, 101069. [CrossRef]
6. Wang, X.; Petriaev, A.; Kolos, A.; Chetina, M. Prediction of thermal regime and frost heave deformations of subgrade soils during non-stationary heat and moisture transfer. *Transp. Geotech.* **2024**, *48*, 101338. [CrossRef]
7. Ulitsky, V.; Gorodnova, E. The construction of transport infrastructure on permafrost soils. *Procedia Eng.* **2017**, *189*, 421–428. [CrossRef]
8. Haghi, N.; Hashemian, L.; Bayat, A. Effects of seasonal variation on the load-bearing capacity of pavements composed of insulation layers. *Transp. Res. Rec.* **2016**, *2579*, 87–95. [CrossRef]
9. Wang, Q.; Fang, J.; Zhao, X.; Hu, K. The influence of pavement type on the thermal stability of block-stone embankments in the warm permafrost region. *Transp. Geotech.* **2020**, *23*, 100334. [CrossRef]
10. Huang, Y.; Niu, F.; He, P.; Pan, W.; Yuan, K.; Su, W. Effectiveness evaluation of cooling measures for express highway construction in permafrost regions based on GPR and ERT. *Cold Reg. Sci. Technol.* **2024**, *228*, 104339. [CrossRef]
11. Hu, T.; Cui, C.; Yue, Z.; Liu, B.; Qi, L.; Cai, D. Literature review of prevention and control technologies of thermal thawing damage in subgrades of permafrost regions. *China Rai. Sci.* **2025**, *46*, 1–17. Available online: <https://zgk.publish.founderss.cn/thesisDetails#10.3969/j.issn.1001-4632.2025.02.01&lang=zh> (accessed on 2 September 2025).
12. Yuan, K.; Zhang, J.; Fan, K.; Zhu, D. Application effect and numerical simulation of hollow block ventilated roadbed in frozen soil areas. *Hig. Transp. Technol.* **2013**, *30*, 56–62. [CrossRef]
13. Cheng, Z.; Zhang, Z.; Liu, G.; Wu, W. Study on the effect of non-linear ventilation pipe composite measures on the slope permafrost subgrade. *Sustainability* **2023**, *15*, 1180. [CrossRef]
14. Jørgensen, A.; Ingeman-Nielsen, T. Optimization in the use of air convection embankments for the protection of underlying permafrost. *Cold Reg. Eng.* **2012**, *10*, 12–20. [CrossRef]
15. Li, Y.; Wu, Z.; Wang, Y.; Sun, J. Test study on the application effect of the thermal pipes on the roadbed in the permafrost region along Qinghai-Tibet Railway. *China Rai. Sci.* **2008**, *29*, 6–11. [CrossRef]
16. Liu, J.; Li, Z.; Li, Y.; Zhou, D. Research on the cooling control efficiency of hot rod roadbeds in permafrost regions. *Highway* **2021**, *66*, 20–27. Available online: <https://link.cnki.net/urlid/11.1668.U.20210202.1606.008> (accessed on 7 May 2024).
17. Tai, B.; Liu, J.; Fang, J.; Xu, A.; Liu, L. Calculation model of permafrost table of XPS insulated board subgrade in warm permafrost regions. *J. Chang'an Univ.* **2017**, *37*, 1–11. Available online: <https://zzszrb.chd.edu.cn/oa/DArticle.aspx?type=view&id=201705026> (accessed on 9 June 2023).
18. Zhang, B.; Fan, W. Application of insulation board in road engineering in permafrost regions of Inner Mongolia. *J. Geomech.* **2018**, *24*, 706–713. [CrossRef]
19. Chen, D.; Wang, S.; Chen, J.; Dong, Y. Study of the factors influencing the thickness of residual thawed interlayers and cooling effect of block-stone embankment. *J. Glaciol. Geocryol.* **2014**, *36*, 854–861. [CrossRef]
20. Ma, M.; Qi, Y.; Guo, Y.; Dong, Y. Numerical simulation of influence of different replacement materialson highway subgrade temperature in permafrost area. *For. Eng.* **2022**, *4*, 131–139. [CrossRef]
21. Goering, D.J. Passively cooled railway embankments for use in permafrost areas. *J. Cold Reg. Eng.* **2003**, *17*, 119–133. [CrossRef]
22. Pei, W.; Zhang, M.; Li, S.; Lai, Y.; Jin, L. Enhancement of convective cooling of the porous crushed-rock layer in cold regions based on experimental investigations. *Int. Commun. Heat Mass Transfer.* **2017**, *87*, 14–21. [CrossRef]

23. Shang, Y.; Cao, Y.; Li, G.; Gao, K.; Zhang, H.; Sheng, J.; Chen, D.; Lin, J. Characteristics of meteorology and freeze-thaw in high-latitude cold regions: A case study in Da Xing'anling, Northeast China (2022–2023). *Front. Earth Sci.* **2025**, *12*, 1476234. [CrossRef]
24. Anderson, J.; Douglas, T.; Barbato, R.; Saari, S.; Edwards, J.; Jones, R. Linking vegetation cover and seasonal thaw depths in interior Alaska permafrost terrains using remote sensing. *Remote Sens. Environ.* **2019**, *233*, 111363. [CrossRef]
25. Szymański, W.; Jagi, K.; Drewnik, M.; Musielok, Ł.; Stolarczyk, M.; Gus-Stolarczyk, M.; Sikora, S. Impact of tundra vegetation type on topsoil temperature in central Spitsbergen (Svalbard, High Arctic). *Geoderma* **2022**, *428*, 116196. [CrossRef]
26. Heijmans, M.M.P.D.; Magnússon, R.Í.; Lara, M.J.; Frost, G.V.; Myers-Smith, I.H.; van Huissteden, J.; Jorgenson, M.T.; Fedorov, A.N.; Epstein, H.E.; Lawrence, D.M.; et al. Tundra Vegetation Change and Impacts on Permafrost. *Nat. Rev. Earth. Environ.* **2022**, *3*, 68–84. [CrossRef]
27. Grünberg, I.; Wilcox, E.; Zwieback, S.; Marsh, P.; Boike, J. Linking tundra vegetation, snow, soil temperature, and permafrost. *Biogeosciences* **2020**, *17*, 4261–4279. [CrossRef]
28. Chang, X.; Jin, H.; Zhang, Y.; Li, X.; He, R.; Li, Y.; Lü, L.; Wang, H. Permafrost thermal dynamics at a local scale in northern Da Xing'anling Mountains. *Environ. Res. Lett.* **2024**, *19*, 064014. [CrossRef]
29. Lara, R.; Berg, A.; Warland, J.; Tetlock, E. In situ estimates of freezing/melting point depression in agricultural soils using permittivity and temperature measurements. *Water Resour. Res.* **2020**, *56*, 5. [CrossRef]
30. Watanabe, K.; Osada, Y. Simultaneous measurement of unfrozen water content and hydraulic conductivity of partially frozen soil near 0 °C. *Cold Reg. Sci. Technol.* **2017**, *142*, 79–84. [CrossRef]
31. Li, C.; Vennapusa, P.K.R.; Ashlock, J.; White, D.J. Mechanistic-based comparisons for freeze-thaw performance of stabilized unpaved roads. *Cold Reg. Sci. Technol.* **2017**, *141*, 97–108. [CrossRef]
32. Skierucha, W.; Wilczek, A.; Szyplowska, A.; Sławiński, C.; Lamorski, K. A TDR-based soil moisture monitoring system with simultaneous measurement of soil temperature and electrical conductivity. *Sensors* **2012**, *12*, 13545–13566. [CrossRef] [PubMed]
33. Teltayev, B.; Oliviero Rossi, C.; Aitbayev, K.; Suppes, E.; Yelshibayev, A.; Nugmanova, A. Freezing and Thawing Processes of Highways in Kazakhstan. *Appl. Sci.* **2022**, *12*, 11938. [CrossRef]
34. Hu, G.; Zhao, L.; Zhu, X.; Wu, X.; Wu, T.; Li, R.; Xie, C.; Hao, J. Review of algorithms and parameterizations to determine unfrozen water content in frozen soil. *Geoderma* **2020**, *368*, 114277. [CrossRef]
35. Bai, Q.; Li, X.; Tian, Y.; Fang, J. Equations and numerical simulation for coupled water and heat transfer in frozen soil. *J. Geotech. Eng.* **2015**, *37*, 131–136. [CrossRef]
36. Xu, Z.; Ma, M.; Chen, Y.; Shan, W.; Wu, K.; Ji, J. Study on hydrothermal migration and deformation characteristics of black soil during freeze-thaw processes driven by multi-factor couplings. *Soils* **2025**, *57*, 1152–1167. [CrossRef]
37. Li, S.; Zhang, M.; Pei, W.; Lai, Y. Experimental and numerical simulations on heat-water-mechanics interaction mechanism in a freezing soil. *Appl. Therm. Eng.* **2018**, *132*, 209–220. [CrossRef]
38. Wilczek, A.; Szyplowska, A.; Kafarski, M.; Skierucha, W. A time-domain reflectometry method with variable needle pulse width for measuring the dielectric properties of materials. *Sensors* **2016**, *16*, 191. [CrossRef]
39. Chuvilin, E.; Davletshina, D.; Bukhanov, B.; Grebenkin, S. Thermal conductivity of frozen and unfrozen gas-saturated soils. *Geosciences* **2023**, *13*, 347. [CrossRef]
40. Fu, J.; Zhu, D.; Zhang, H. Research on special subgrade base treatment method for Gonghe-Yushu Highway in permafrost regions. *Highway* **2016**, *1*, 36–42. Available online: <https://xueshu.baidu.com/ndscholar/browse/detail?paperid=1698d9e2a213cc98c28c9fe22f649af1> (accessed on 4 March 2024).
41. Liu, M.; Ma, W.; Niu, F.; Luo, J.; Yin, G. Thermal performance of a novel crushed-rock embankment structure for expressway in permafrost regions. *Int. J. Heat Mass Transf.* **2018**, *127*, 1178–1188. [CrossRef]
42. Wu, Q.; Zhao, H.; Zhang, Z.; Chen, J.; Liu, Y. Long-term role of cooling the underlying permafrost of the crushed rock structure embankment along the Qinghai–Xizang railway. *Permafr. Periglac. Process.* **2020**, *31*, 172–183. [CrossRef]
43. Li, X.; Ma, W.; Mu, Y.; Dong, J.; Chen, L. Wind field and thermal performances of an expressway constructed with two separated crushed-rock embankments in high-altitude permafrost zones. *Transp. Geotech.* **2021**, *27*, 100447. [CrossRef]
44. Tian, Y.H.; Fang, J.H.; Shen, Y.P. Simulation on stability of large-width highway embankment in permafrost regions. *China J. Highw. Transp.* **2015**, *28*, 17–23. [CrossRef]
45. Shan, W.; Ma, M.; Guo, Y.; Zhang, C. Numerical analysis of the influence of block-stone embankment filling height on the water, temperature, and deformation distributions of subgrade in permafrost regions. *Water* **2022**, *14*, 1382. [CrossRef]
46. Schuur, E.A.G.; McGuire, A.D.; Schädel, C.; Grosse, G.; Harden, J.W.; Hayes, D.J.; Hugelius, G.; Koven, C.D.; Kuhry, P.; Lawrence, D.M.; et al. Climate change and the permafrost carbon feedback. *Nature* **2015**, *520*, 171–179. [CrossRef]
47. Peng, X.; Zhang, T.; Frauenfeld, O.W.; Wang, K.; Cao, B.; Zhong, X.; Su, H.; Mu, C. Response of seasonal soil freeze depth to climate change across China. *Cryosphere* **2017**, *11*, 1059–1073. [CrossRef]

48. Zhang, S.; Chen, J.; Huo, L.; Li, X.; Wu, C.; Zhang, H.; Feng, Q. The response of alpine permafrost to decadal human disturbance in the context of climate warming. *Remote Sens.* **2025**, *17*, 3482. [CrossRef]
49. Wang, Y.; Zhao, Y.; Mao, X.; Yin, S. Impact of climate change on the performance of permafrost highway subgrade reinforced by concrete piles. *Future Transp.* **2023**, *3*, 996–1006. [CrossRef]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Investigation of Seepage Behavior and Settlement Deformation Mechanisms in Loess Embankment Foundation Systems in Eastern Gansu Province

Wei Wang ¹, Wei Li ¹, Pengxiang Zhang ¹ and Lulu Liu ^{2,*}

¹ Jinan Huanghe Road and Bridge Construction Group Co., Ltd., Jinan 250099, China; warren1080@126.com (W.W.); liwei9890816@163.com (W.L.); 18535003000@163.com (P.Z.)

² School of Mechanics and Civil Engineering, China University of Mining and Technology, Xuzhou 221116, China

* Correspondence: believeliululu@163.com or liululu@cumt.edu.cn; Tel.: +86-18651611376

Abstract: The northwestern region of China is characterized by loess soil and seasonal permafrost. Due to the combined effects of its unique climate and precipitation patterns, local roads frequently suffer from issues such as foundation settlement, erosion, and collapse, which pose significant risks to both road construction and safe operation. This study examines a typical high subgrade in Northwest China, where a scaled laboratory model experiment was conducted. The research investigates the impact of water infiltration at the slope foot, under the dual influences of extreme cold and precipitation, on changes in the internal moisture field and settlement deformation characteristics of both the foundation and subgrade. The results indicate that the variation in moisture content across the section follows an arc-shaped diffusion pattern. Settlement is influenced by both the amount of infiltrated water and cold air, with a noticeable lag effect. A settlement of 0.1 cm is considered the threshold for significant impact, with the minimum observed lag period approaching 4 days. The settlement is concentrated in the slope region, exhibiting a bending failure pattern. Numerical simulations reveal that the cross-sectional settlement distribution forms an inverted “S” shape, and the cumulative moisture content at each monitoring point exhibits a quadratic relationship with the cumulative settlement. The findings of this study provide scientific guidance and technical references for road construction and safe operation in the seasonal permafrost regions of Northwest China.

Keywords: loess; permafrost; precipitation; subgrade; foundation; moisture field; settlement deformation

1. Introduction

The condition of loess subgrade is primarily influenced by both human and natural factors. Among the natural factors, infiltration caused by atmospheric precipitation, slope collapse, pavement cracking, and similar issues are key contributors [1–4]. Subgrades, particularly under the combined influence of external factors such as load, rainfall, and temperature, are prone to various forms of damage, including pits, arches, joint steps, and settlement at the bridge head or culvert ends [5–7]. These issues significantly impair the normal functioning of highways, leading to substantial economic losses and even casualties. Infiltration due to external precipitation is one of the most severe factors, particularly at the foot and side slopes [8,9].

Loess, a common material used for subgrade filling, undergoes significant structural changes and strength reduction after flooding, which poses substantial safety risks to road

construction and operation [10–13]. As a result, scholars have conducted extensive research on the characteristics of loess subgrade following water seepage. Jiang et al. [14] investigated the water stability and permeability of loess at varying compaction levels. The results indicate that increasing the compaction degree significantly enhances the loess subgrade's resistance to water infiltration. Liu et al. [15] proposed a theoretical framework for loess vibration permeability and elucidated the mechanism by which vibration acceleration affects loess subgrade permeability. Zhang et al. [16] developed a custom soil–water characteristic curve (SWCC) measurement system and an enhanced permeability testing device and conducted tests on silty clay compacted samples. The effects of overlying stress and dry–wet cycles on the hydraulic characteristics of the subgrade, particularly the SWCC and saturation permeability coefficient, were analyzed, and a corresponding fitting model was established. Finally, the subgrade water transfer characteristics, considering the influences of dry–wet cycles and overlying stress, were analyzed using the finite element method. Yue et al. [17] investigated the effects of an amide polymer (AP) on the permeability, mechanical properties, and crack resistance of loess through an orthogonal test. The basic properties of the AP, along with the permeability, mechanical properties, and dry–wet variation characteristics of polymer-modified loess, were tested, and scale model verification and simulation analysis were performed. Sim et al. [18] examined the influence of groundwater level and rainfall infiltration on the stability of the test subgrade and noted that future research should explore the consolidation process following a decrease in the groundwater level. Liu et al. [19] conducted long-term monitoring of the humidity within a subgrade constructed from recycled construction waste on the Xi'an-Xianyang expressway in China. The dynamics of the road axis, lane, soil shoulder, hard shoulder, and the under-slope of the roadblock and arc protection slope were analyzed in detail.

In seasonal frozen soil areas, where rainfall and groundwater influence the environment, excessively wet road surfaces are more likely to induce subgrade frost heave and settlement deformation. The northwestern region is a typical loess area and also a seasonal frozen soil zone. Due to extreme climate conditions, snow tends to accumulate on the slope and foot of the subgrade in the winter. As temperatures rise, rain and snow melt, infiltrating the subgrade and causing localized wetting and softening, which pose significant risks to slope stability and road safety [20–22]. Zhao et al. [23] examined the effect of soil texture on the infiltration of snowmelt into frozen soil. The mechanical properties of infiltration processes in clay, silty clay loam, and sandy loam were studied using a physics-based numerical simulation of the frozen soil phase transition and heat transfer. Stuurop et al. [24] simulated snowmelt infiltration into frozen soil using a numerical model to investigate the theoretical controls on frozen soil infiltration across several soil types. Wen et al. [25] systematically studied the effects of water immersion, freeze–thaw cycles, and water and salt transport on the road characteristics of mine overburden with varying coarse-grain contents and proposed a cumulative damage model for mine overburden.

Previous studies have primarily focused on the permeability behavior of subgrade slopes, with research largely limited to the analysis of the subgrade's moisture field and settlement deformation. However, water accumulation at the foot of high-fill subgrades is a critical factor contributing to slope instability and foundation degradation. Therefore, conducting water seepage tests in ditches at the foot of the subgrade and simulating the coupling relationship between the moisture field and settlement deformation within the subgrade and foundation is of significant importance. This article examines a representative high-fill loess subgrade in Northwest China through scaled model testing, investigating how localized water seepage in slope foot ditches affects the subgrade's internal moisture distribution and settlement deformation characteristics. The research also explores the impact of snowmelt on the loess subgrade in the northwestern seasonal freezing zone. The

findings aim to provide a scientific basis and academic reference for road construction and operation in this region. The overall methodology and idea are shown in Figure 1.

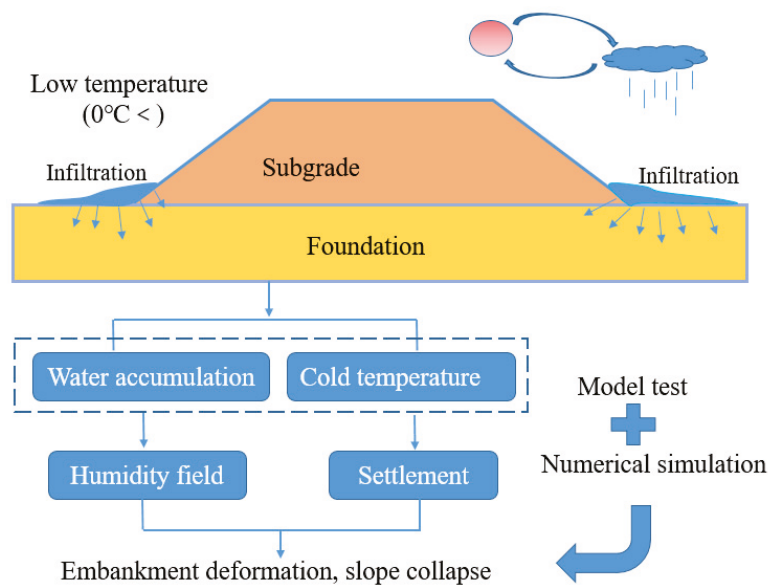


Figure 1. The overall methodology and idea.

2. Test Design and Scheme

2.1. Supporting Conditions

The project area is located in the loess plateau, a region characterized by a complex landform with numerous gullies; here, the average annual rainfall is 300–500 mm, the average annual temperature is 5°C , and the average winter temperature is -6.9°C . Under the influence of flowing water, the loess is typically eroded, forming features such as loess beams and loess hills. The area between Liang and Mao is primarily composed of narrow, deeply cut loess gullies, with bedrock often exposed. In addition to these narrow gullies, erosive valley landforms are also present, which are relatively flat and wide; field compaction of the subgrade and the landform characteristics are shown in Figure 2. The site consists of self-weight collapsible loess with a medium collapsibility degree [26]. The standard depth of the frozen soil is 820 mm (average depth), and the compaction degree of the foundation is 90%, while the lower subgrade has a compaction degree of 93%, and the upper subgrade has a compaction degree of 95%.



Figure 2. On-site subgrade construction. (a) Subgrade compaction; (b) terrain.

The model test was conducted based on the high-fill subgrade of an engineering test section, using field-disturbed loess as the filler. The specific physical and mechanical properties are provided in Table 1, and the particle size distribution curve and compaction curve of the fill material are shown in Figure 3. The dimensions of the site subgrade and

its schematic diagram are presented in Figure 4. The slope ratio of the upper and lower sections is approximately 1:1.5.

Table 1. Physical and mechanical indicators of disturbed loess at study site.

Soil Sample	Density ρ (g/cm ³)	Dry Density ρ_d (g/cm ³)	Natural Moisture Content ω (%)	Plastic Limit ω_p (%)	Liquid Limit ω_L (%)	Optimum Moisture Content ω_{op} (%)	Maximum Dry Density ρ_{dmax} (g/cm ³)
Loess	1.77	1.69	9.5	9	25	11.2	1.882

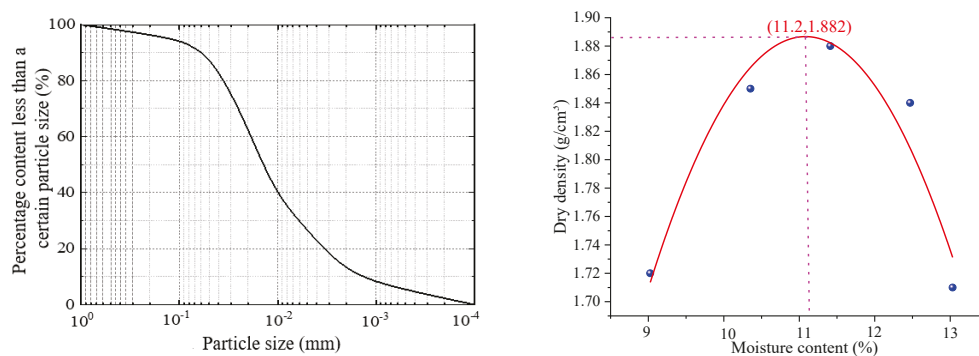


Figure 3. The particle size distribution curve and compaction curve of the fill material.

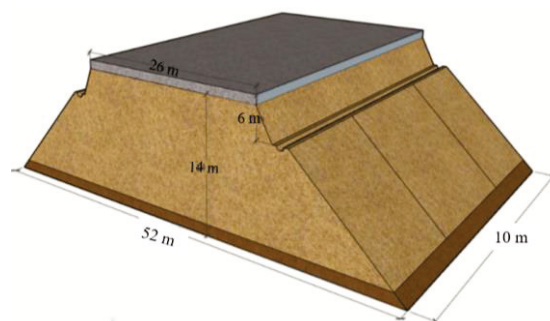


Figure 4. Three-dimensional schematic diagram of on-site subgrade.

2.2. Similarity Ratio

To improve the accuracy of the experiment, the model similarity ratio was maximized. Additionally, considering the influence of the test site and the model boundary effect, the similarity ratio was set at 1:20.

$$\text{Similar in length : } C_l = \frac{l_p}{l_m} \quad (1)$$

$$\text{Similar in volume : } C_v = \frac{V_p}{V_m} = C_l^3 \quad (2)$$

Note: C_l —length similarity ratio (unit: 1); C_v —volume similarity ratio (unit: 1);

l_p —prototype size (unit: m); l_m —model size (unit: cm);

V_p —prototype volume (unit: m³); V_m —model volume (unit: cm³).

Gravity similarity ratio and strength similarity ratio: According to the experiences of previous studies [27], the weights of similar materials can be controlled in a large range. For the convenience of calculation, the similarity ratio of a heavy weight, temperature, and time between the prototype and the model is set to 1.

According to the similarity criterion, the similarity constant of the strength index (including the elastic modulus, cohesion, and compressive strength) is equal to the product of the geometric similarity constant C_l and the heavy similarity constant C_γ ; that is, $C_\sigma = C_\gamma \cdot C_l$.

2.3. Test Equipment and Model Building

2.3.1. Test Equipment

The test equipment includes a soil moisture sensor, a custom settlement measuring rod, a seepage channel, a total station, and an acquisition instrument.

The soil moisture sensor model is RS485. The settlement measuring rod and seepage pipe were custom-made. The settlement measuring rod consists of a steel rod with a diameter of 0.8 cm and a 2 cm diameter disc welded at the bottom. The lengths of the rods were 100 cm (4 rods), 80 cm (4 rods), and 60 cm (4 rods), which were buried at the corresponding positions of each layer sensor. A reflection patch was attached to the top of the settlement measuring rod. The total station was used to monitor the settlement changes at each point during the experiment in real time. Figure 5 shows the schematic of the custom-made settlement measuring rod, seepage pipe, and total station. The pore water pressure data acquisition instrument used in the model test is JC-YBY-2001, produced by the Tokyo Institute of Measurement, Japan.

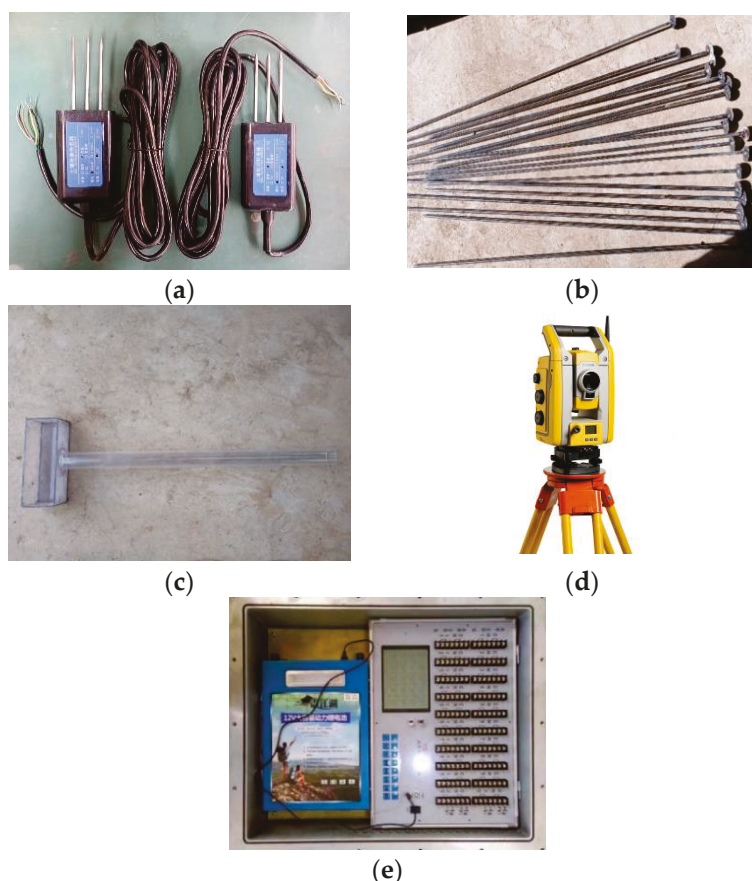


Figure 5. Test equipment. (a) Moisture sensor; (b) settlement measuring rod; (c) seepage pipe; (d) total station; (e) acquisition instrument.

Tables 2 and 3 show the technical indicators of the soil moisture sensor and JC-YBY-2001 data acquisition instrument, respectively.

Table 2. Technical indicators of soil moisture sensor.

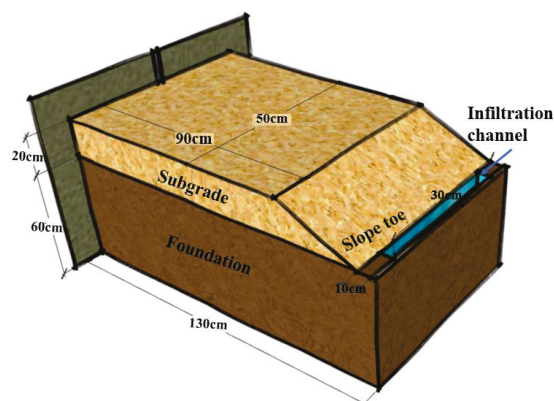
Type	Range (%)	Measurement Accuracy (%)	Response Time (s)	Operating Environment (°C)
RS485	100	±1	<1	−40~80 °C

Table 3. JC-YBY-2001 data acquisition parameters.

Frequency (Hz)	Sensor	Number of Channels	Acquisition Interval (min)	Connection Mode
10	Pore water pressure gauge	20	10	Full bridge

2.3.2. Model Construction and Sensor Layout

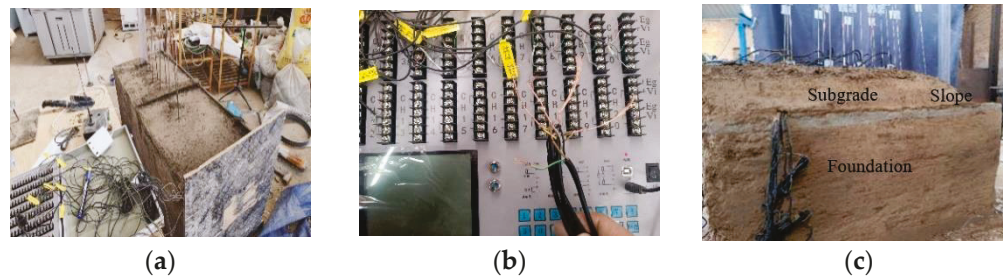
Soil samples from the site were collected, transported, and dispersed before being air-dried to prepare the models, following compaction standards that varied according to the positions of the subgrade and foundation. Due to limitations in the available test soil and site conditions, the road was treated symmetrically along the centerline. Additionally, considering the infinite extension of the foundation and a similarity ratio of 20, the foundation had a height of 0.6 m, a width of 0.5 m, and a length of 1.3 m. Considering that the infiltration from the side ditch primarily humidified the lower subgrade, the height of the far subgrade was controlled at 4 m. After this control, the model subgrade had a height of 0.2 m, with a length of 1.2 m and a width of 0.5 m. A 0.1 m slope foot was reserved, and the subgrade side slope was approximately 1:1.5. The model diagram is shown in Figure 6.

**Figure 6.** Schematic diagram of the model.

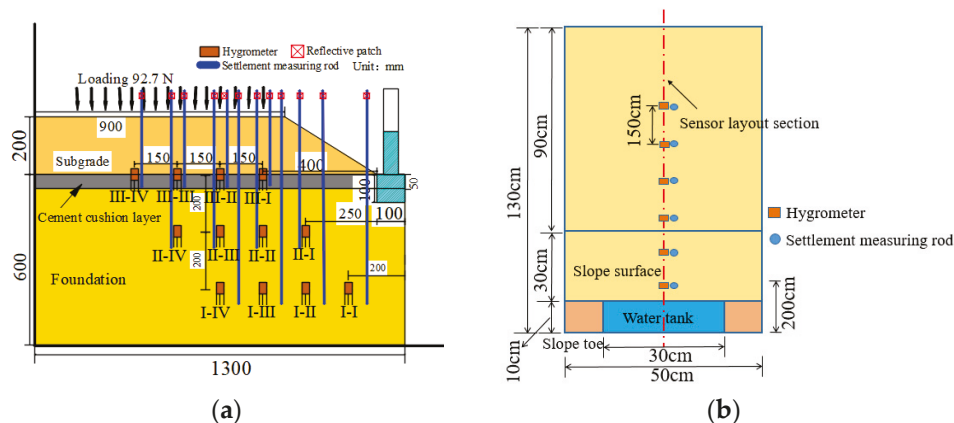
To facilitate a more effective comparison of field construction technologies, a 5 cm cement cushion layer is placed at the interface between the model foundation and subgrade. Each layer is controlled to a height of 3 cm, and a rubber hammer is used as the rolling tool. During the rolling process, water is sprayed onto the surface of each layer. Simultaneously, after each layer is rolled, ring knife samples are collected to measure the moisture content and dry density, ensuring that the physical parameters of the produced model closely match those of the prototype compacted soil. The dry density error must not exceed 0.3 g/cm³, and the moisture content error must not exceed 1%. The specific rolling indicators are presented in Table 4. Based on the road engineering standards and the similarity ratio discussed in this paper, the self-weight of the upper subgrade of the external load model, set at 92.7 N at the top of the model, is determined. The model construction is illustrated in Figure 7.

Table 4. Model-making metrics.

Type	Layer	Thickness	Dry Density	Compactness	Moisture Content	Compaction Method	Height
Foundation	1–20	3 cm	1.69 g/cm ³	90%	11.2%	Rubber hammer	60 cm
Subgrade	1–6	3 cm	1.77 g/cm ³	93%	11.2%	compaction	18 cm
	7	2 cm	1.77 g/cm ³		11.2%		2 cm

**Figure 7.** Model-making process. (a) Sensor embedding; (b) marking; (c) model.

A total of three sensor layers are embedded into the model. A soil hygrometer and settlement measuring rod are, respectively, placed at the foundation heights of 20 cm and 40 cm, as well as at the interface between the subgrade and the foundation. A schematic diagram of sensor embedding is shown in Figure 8. The water channel measures 40 cm in length, 10 cm in width, and 10 cm in height. A water injection pipe is installed in the channel to regularly fill it, simulating the humidity and settlement deformation characteristics of the foundation and subgrade during the infiltration of side ditch water in the cold season. The test dates were 13 December 2023–22 December 2023, the water injection time was 8:00 a.m. every day, and the water injection volume was 2 L. To prevent freezing before infiltration, warm water at 30 °C is used for injection.

**Figure 8.** Schematic diagram of sensor layout. (a) Cross-section; (b) plan.

3. Results and Analysis

The moisture field and settlement deformation characteristics of the foundation and lower subgrade in the loess subgrade of seasonal freezing areas, influenced by water infiltration from the slope foot ditch under atmospheric precipitation, were studied by regularly injecting water into the model's side ditch. Figure 9 illustrates the changes in atmospheric and indoor temperatures.

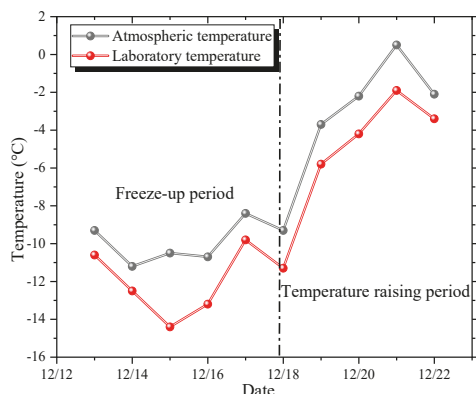


Figure 9. Temperature curve.

As shown in the figure, the indoor temperature closely mirrors the atmospheric temperature, with a significant rise occurring since 15 December. Throughout the monitoring period, the temperatures remain within the freezing range, and the minimum indoor temperature reaches -14.9°C .

3.1. Analysis of the Influence of Water Injection on Settlement and Moisture Content

Figure 10 illustrates the relationship between moisture content, settlement, and the amount of water injected at different monitoring locations of the model.

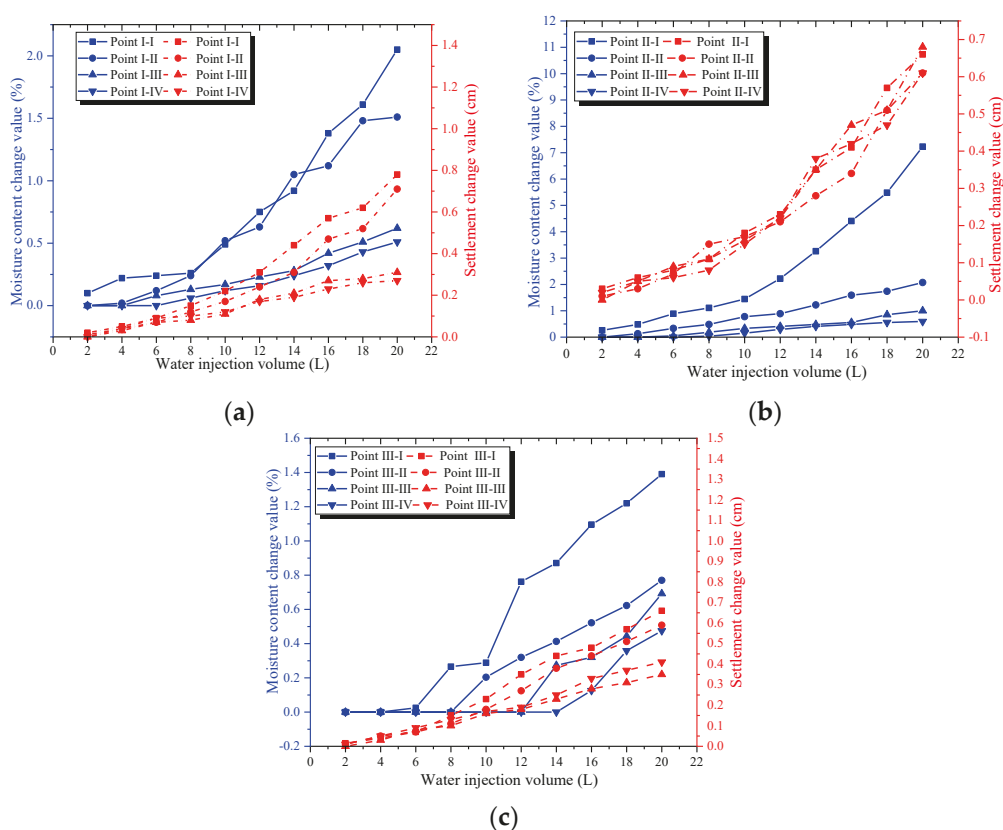


Figure 10. Sedimentation and moisture content with water injection. (a) At a depth of 40 cm in the foundation; (b) at a depth of 20 cm in the foundation; (c) interface between the subgrade and foundation.

As shown in Figure 10, both the moisture content and settlement deformation increase with increasing water injection. The most significant change in moisture content occurs at a depth of 20 cm, where the change between points II-I is the greatest, reaching 7.3%.

This is because the monitoring point is closest to the side ditch and is most affected by water flow infiltration. At a depth of 40 cm and at the interface between the subgrade and foundation, changes in the moisture content are less affected by water injection due to the soil compaction, which reduces the pore ratio and inhibits water flow spread. Furthermore, the primary direction of seepage is vertically downward due to gravity. Settlement deformation at each layer shows that the closer the horizontal distance to the side ditch, the more pronounced the settlement. The settlement at 40 cm ranges from 0.23 to 0.81 cm, at 20 cm from 0.62 to 0.69 cm, and at the interface between the subgrade and the foundation from 0.32 to 0.68 cm. The settlement at a depth of 20 cm is most strongly influenced by water injection.

3.2. Settlement Changes with Time

As shown in Figure 11, the most significant settlement deformation occurs at depths of 40 cm and 20 cm, with a more pronounced settlement observed at points closer to the ditch at the foot of the slope. The maximum settlement, 0.81 cm, occurs at point I-I. The settlement behavior of each layer exhibited slow deformation from December 13 to December 16, followed by an increase in deformation from December 16 to December 23. This trend can be attributed to the fact that during the initial stage of water seepage, the external temperature was at its lowest for the entire test period, causing the water infiltrating into the soil to freeze. Additionally, the infiltration process took time, resulting in small settlement deformations in the early stages of the test. A settlement of more than 0.1 cm is considered the threshold for the effective affected area. The settlement lag time at a depth of 40 cm is approximately 4 days, while at 20 cm, the lag time at points I and II is about 5 days, and at points III and IV, it is approximately 7 days. The lag times for the interface points I and II are around 4 days, while for points III and IV, they are approximately 8 days.

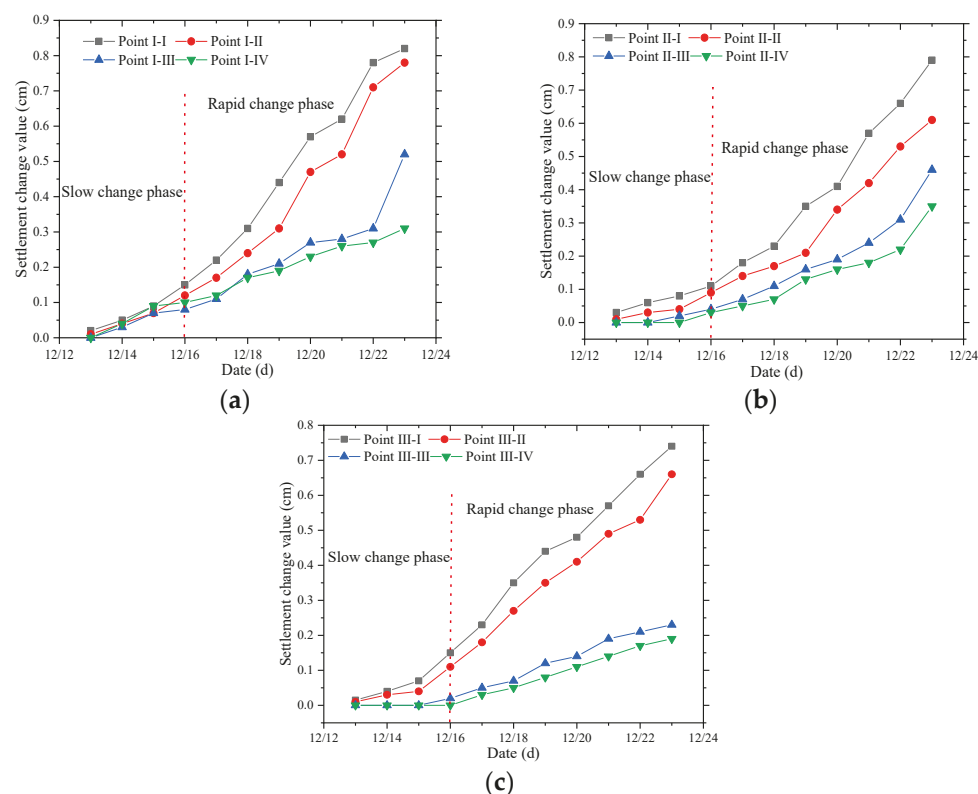


Figure 11. Settlement with time at different locations. (a) At a depth of 40 cm in the foundation; (b) at a depth of 20 cm in the foundation; (c) interface between the subgrade and foundation.

By comparing Figure 11a–c, it is evident that the settlement at points I and II of each layer is most pronounced, with settlement exceeding 0.5 cm by the end of the test. In contrast, the settlement deformation at points III and IV, located at the interface between the subgrade and foundation, is minimal. This is attributed to the distance of the monitoring area from the water injection point, which limits the spread of water flow. The results indicate that the wetting and softening damage caused by infiltration from the side ditch of the fill subgrade primarily affects the foundation. Overall, the model exhibits a “bending dump” failure due to water seepage.

3.3. Finite Element Settlement Simulation Based on Plaxis

In this section, the settlement of the model was simulated using Plaxis (2024) finite element software. The HS-Small model was employed for calculations during the model creation. The soil layers were classified based on their properties and mechanical parameters, as outlined in the survey report, with similar properties grouped together to simplify the calculation. The permeability of the unsaturated subgrade soil is treated as a transient saturation–unsaturated soil seepage problem. Assuming that the soil skeleton does not deform and that water is an incompressible fluid, the governing equation for the two-dimensional plane problem is expressed as follows:

$$\frac{\partial}{\partial x}(K_x(\theta)\frac{\partial H}{\partial x}) + \frac{\partial}{\partial y}(K_y(\theta)\frac{\partial H}{\partial y}) = \rho_w g m_2^w \frac{\partial H}{\partial t} \quad (3)$$

Note: ρ_w —water density; $K_x(\theta)$, $K_y(\theta)$ —permeability coefficient of unsaturated soil in the x and y directions; H —total head; θ —volumetric water content of unsaturated soil; m_2^w —coefficient of volume change ($m_2^w = -\frac{\theta}{\psi}$, $\psi = u_a - u_w$); g —gravitational acceleration; u_a —pore air pressure; u_w —pore water pressure.

The governing equation is discretized using the finite element method, and time integration is performed using the central difference method. Since infiltration does not affect the model’s periphery, it is assumed that the horizontal displacement at the model boundary is zero, while deformation is permitted in the numerical direction. Additionally, deformation in any direction at the lower boundary is constrained to zero. The physical and mechanical parameters of each soil layer are provided in Table 5. Settlement deformations at the mid-point of seepage (December 18) and at the end of seepage (December 23) were simulated to verify the accuracy of the test.

Table 5. Physical and mechanical parameters of this model.

Type	Thickness (cm)	Effective Cohesion c (kPa)	Effective Angle of Internal Friction φ (°)	Gravity γ (kN/m ³)	Elastic Modulus E (MPa)	Poisson’s Ratio ν	Saturated Permeability Coefficient (cm/s)	Permeability Coefficient (cm/s)
Foundation	0–60	19.8	32.9	16.9	8	0.33	5.41×10^{-6}	3.93×10^{-6}
Subgrade	0–20	19.8	32.9	17.5	8	0.33	1.12×10^{-7}	8.67×10^{-7}

The model grid is divided into 11,660 units as shown in Figure 12.

Finite element simulations reveal that settlement primarily occurs in the middle and late stages of water seepage, with an inverted “S” contour distribution. Settlement deformation is more pronounced the closer the location is to the side ditch. This is due to the significant increase in soil moisture content in this area as a result of water seepage, causing the soil mass to collapse under its own weight. During the middle stage of seepage, settlement deformation is relatively small, with the maximum value reaching 0.50 cm near the foot of the slope. By the end of seepage, most of the model’s settlement exceeds 0.40 cm, with the maximum value reaching 1.0 cm, primarily at the side slope and slope foot. A

comparison of Figure 13a,b shows that the settlement is predominantly concentrated at the slope foot and in the slope area. This is due to the generation of high pore water pressure and dynamic water pressure at the slope foot, which, combined with water seepage, causes the soil to gradually transition to a wet and soft state. As a result, the slope progressively collapses under its own gravity. The results indicate that water infiltration at the foot of the slope causes significant damage to the foundation and subgrade, leading to considerable settlement deformation. In practical engineering, strengthening slope support and drainage at the foot of the slope is essential.

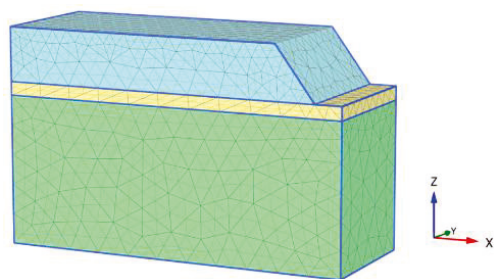


Figure 12. Model meshing.

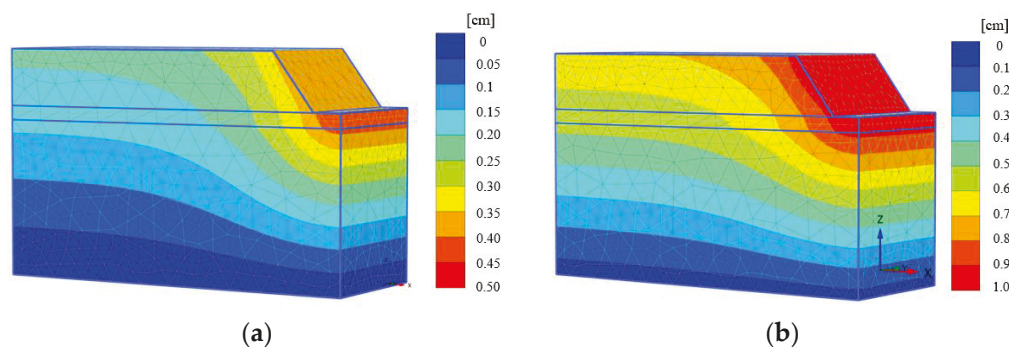


Figure 13. Model settlement contour diagram. (a) Mid-phase seepage; (b) later-phase seepage.

Based on the measured data from the model test, the settlement at each monitoring point at a depth of 40 cm ranges from 0.27 to 0.82 cm, while at a depth of 20 cm, the settlement ranges from 0.35 to 0.80 cm. The settlement at the interface between the foundation and the subgrade varies between 0.15 and 0.75 cm.

3.4. Change of Section Moisture Content

In this section, Surfer (15) software is used to analyze the humidity of the embedded sensor section in the model, and the change trend of the moisture content of the section in the middle period of seepage (18 December) and at the end of seepage (23 December) is studied, respectively.

As can be seen from Figure 14a, at the initial stage of seepage, the water flow mainly diffuses horizontally. If a change of 1% in the moisture content is the effective influence area, the influence area is mainly in the circular arc area with the slope foot as the center, where the radius is 40 cm, and the maximum change value reaches 4.2%. As can be seen from Figure 14b, with continuous water injection, the direction of water flow gradually diffuses to all sides, and the influence range approximately presents an arc area with the slope foot as the center of the circle and a radius of 60 cm, with the maximum change value reaching 7.3%. At the end of the test, the bulk moisture content of most regions of the foundation fluctuates, and the bulk moisture content of some of the subgrade fluctuates slightly, which indicates that the water in the ditch at the foot of slope in actual engineering

mainly affects the wetting of the foundation and the interface between the subgrade and the foundation and then induces the collapse of the subgrade slope.

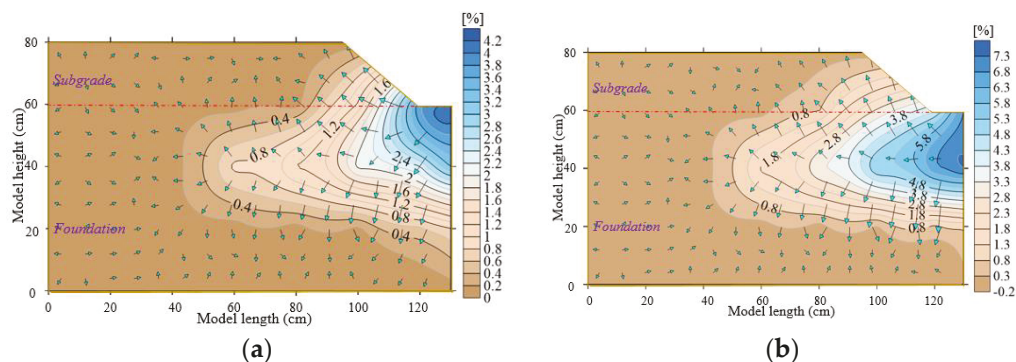


Figure 14. Contour of the moisture content of this model. (a) Mid-phase seepage; (b) later-phase seepage.

By comparison with Figure 14a,b, it can be seen that seepage diffuses in the form of waves, gradually increases with time and temperature, and the diffusion trend is significant. Changes in the moisture content caused by infiltration are mainly concentrated at the foundation, while the subgrade hardly changes.

3.5. Relationship Between Moisture Content and Settlement Deformation

In this section, Surfer software is used to analyze the humidity of the embedded sensor section in the model, and the trend in moisture content at two stages of seepage, the mid-point (18 December) and the end (23 December), is examined. Figure 15 illustrates the changes in moisture content during these two periods. Table 6 shows the settlement fitting equation.

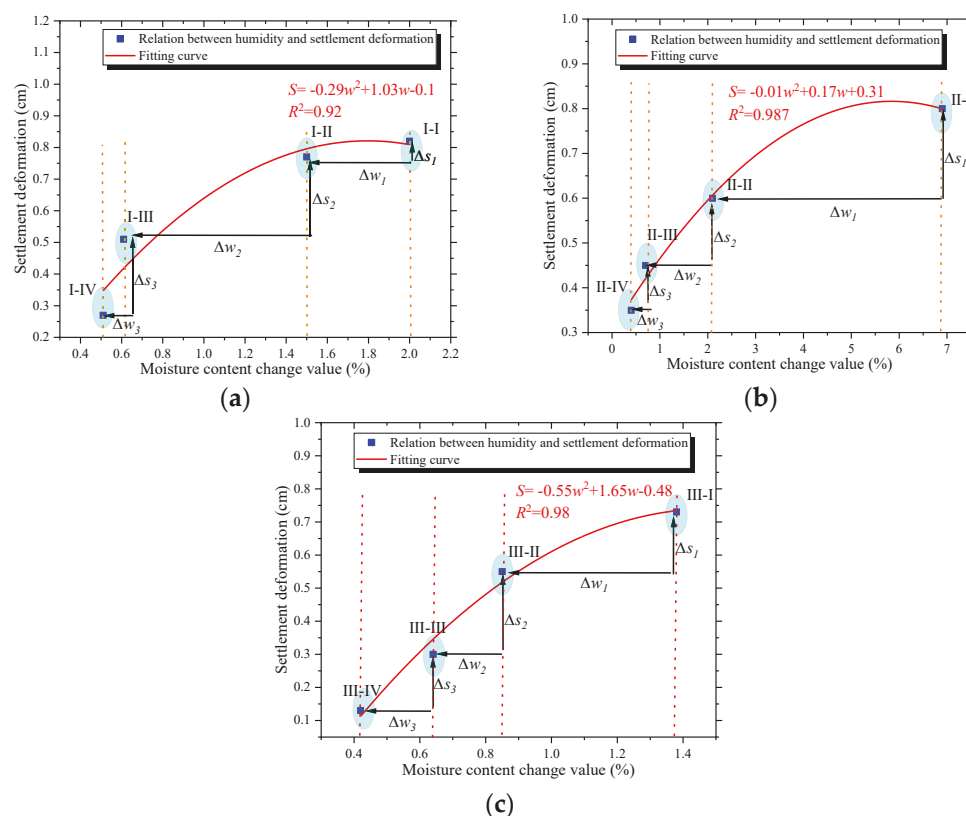


Figure 15. Moisture content and settlement deformation. (a) At a depth of 40 cm in the foundation; (b) at a depth of 20 cm in the foundation; (c) interface between the subgrade and foundation.

Table 6. Settlement fitting equations.

Layer	Fitted Equation	Coefficient of Determination
At a depth of 40 cm in the foundation	$S = -0.29w^2 + 1.03w - 0.1$	$R^2 = 0.92$
At a depth of 20 cm in the foundation	$S = -0.01w^2 + 0.17w + 0.31$	$R^2 = 0.987$
Interface between the subgrade and foundation	$S = -0.55w^2 + 1.65w - 0.48$	$R^2 = 0.98$

As shown in Figure 15a, during the initial stage of seepage, the water flow primarily diffuses horizontally. Defining a 1% change in moisture content as the effective influence area, the impact is mainly concentrated in a circular arc centered at the slope foot with a radius of 40 cm, where the maximum change is 4.2%. As shown in Figure 15b, with continuous water injection, the water flow gradually diffuses in all directions. The influence area expands to an arc with a radius of 60 cm, centered at the slope foot, with the maximum change reaching 7.3%. By the end of the test, the moisture content of most regions of the foundation fluctuates, while some areas of the subgrade show minimal changes. This indicates that in practical engineering, the water in the ditch at the foot of the slope mainly affects the wetting of the foundation and the interface between the subgrade and the foundation, eventually contributing to the collapse of the subgrade slope.

Comparing Figure 15a,b, it is evident that the seepage diffuses in a wave-like pattern, gradually intensifying with time and temperature. The diffusion trend is significant, with changes in the moisture content primarily concentrated at the foundation, while the subgrade remains largely unaffected.

The relationship of accumulated settlement and deformation at each measuring point is shown in Formula (4).

$$S = aw^2 + bw + c \quad (4)$$

where S —cumulative settlement value;

w —cumulative moisture content value;

a , b , and c —undetermined coefficients.

4. Conclusions

This study investigates moisture infiltration induced by atmospheric precipitation in high-fill loess subgrades during cold periods in Northwest China. Through scaled model testing, we analyze the coupled evolution of the subgrade's humidity field and settlement deformation. The main conclusions are as follows:

- (1) The greatest changes in moisture content and settlement occur at a depth of 20 cm. Within the same layer, the settlement and moisture content increase as the distance to the slope foot decreases. Due to the combined effects of water seepage and cold air, the settlement exhibits a clear lag. A settlement greater than 0.1 cm is considered effective. The settlement lag time near the slope foot is approximately 4 days (with a permeability of 8 L).
- (2) The settlement of the subgrade is primarily concentrated in the slope area due to the increased wetness at the foot of the slope, resulting in a “bending tipping failure”. Finite element simulations using Plaxis show that the settlement profile follows an inverted “S” shape.
- (3) A quadratic relationship exists between cumulative settlement deformation and the cumulative moisture content change at each monitoring point. The maximum settlement at each layer is approximately 0.8 cm, occurring near the slope foot.

- (4) The addition of a temperature sensor can be used to study the freezing depth of soil under cold conditions, as well as the relationship between temperature, moisture, and settlement and deformation.
- (5) The incorporation of a slope spraying device can be used to simulate the effects of atmospheric precipitation-induced slope infiltration.

Author Contributions: Conceptualization, W.W. and W.L.; methodology, W.L. and P.Z.; software, W.W. and P.Z.; investigation, P.Z.; resources, P.Z. and L.L.; data curation, W.L.; writing—W.W. and L.L.; original draft preparation, L.L.; writing—review and editing, W.W. and W.L. All authors have read and agreed to the published version of the manuscript.

Funding: This study was funded by the National Natural Science Foundation of China (42302320). Recipient: L.L.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Informed consent was obtained from all subjects involved in the study.

Data Availability Statement: All data, models, and codes generated or used during the study appear in the submitted article.

Acknowledgments: None.

Conflicts of Interest: Author W.W., W.L. and P.Z. were employed by the company Jinan Huanghe Road and Bridge Construction Group Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

References

1. Zhao, M.; Chen, L.; Wu, H.; Guo, W.; Ye, C. Computer microscopic test study on the suitability of modified polyimide grease used to improve collapsible loess railway subgrade. *Concurr. Comput. Pract. Exp.* **2018**, *30*, e4900.
2. Asadi, M.; Mallick, R.; Nazarian, S. Numerical modeling of post-flood water flow in pavement structures. *Transp. Geotech.* **2021**, *27*, 100468.
3. Zhong, X.; Liang, Y.; Wang, Q.; Ma, J.; Liang, S.; Wang, Y.; Xu, X. Evaluation and analysis of the effect of lignin amelioration on loess collapsibility. *J. Renew. Mater.* **2022**, *10*, 3405.
4. Showkat, R.; Mohammadi, H.; Babu, G.S. Effect of rainfall infiltration on the stability of compacted subgrades. *Int. J. Geomech.* **2022**, *22*, 04022104. [CrossRef]
5. Remišová, E.; Decký, M.; Podolka, L.; Kováč, M.; Vondráčková, T.; Bartuška, L. Frost index from aspect of design of pavement construction in Slovakia. *Procedia Earth Planet. Sci.* **2015**, *15*, 3–10.
6. Remišová, E.; Briliak, D. Investigation of the environmental degradation of asphalt pavement mixtures. In Proceedings of the 8th International Conference on Road and Rail Infrastructure, Cavtat, Croatia, 15–17 May 2024.
7. Yao, Y.; Fan, J.; Li, J. A Review of Advanced Soil Moisture Monitoring Techniques for Slope Stability Assessment. *Water* **2025**, *17*, 390. [CrossRef]
8. Tu, G.; Huang, R. Infiltration in two types of subgrades and the effects of rainfall time on the stability of slopes. *Q. J. Eng. Geol. Hydrogeol.* **2016**, *49*, 286–297.
9. Zhou, X.; Zhang, Y.; Li, L.; Zhao, G.; Xu, S. Adhesive performance and impermeability mechanism of loess subgrade using castor oil based polyurethane reinforce. *Constr. Build. Mater.* **2023**, *364*, 130006.
10. Zhao, X.; Zhao, L.; Yang, Q.; Wang, Z.; Cheng, A.; Mo, L.; Yan, J. Permeability and disintegration characteristics of composite improved phyllite soil by red clay and cement. *Minerals* **2022**, *13*, 32. [CrossRef]
11. Ma, B.C.; Tian, W.P.; Li, J.C. Study on Causes and Preventive Measures for Flood Damage of Highway Subgrade along Rivers. *Appl. Mech. Mater.* **2012**, *212*, 748–751.
12. Qiao, Y.; Zhang, S.; Wang, Y.; Dawson, A.; Wake, C.; Ma, T. Simulating floodwater movement in pavements for developing post-flooding time-depth-damage functions. *Constr. Build. Mater.* **2023**, *396*, 132408.
13. Elshaer, M.; Ghayoomi, M.; Daniel, J.S. Impact of subsurface water on structural performance of inundated flexible pavements. *Int. J. Pavement Eng.* **2019**, *20*, 947–957. [CrossRef]
14. Jiang, Y.; Yi, Y.; Tian, T.; Sha, H.; Fan, J.; Ji, X.; Xue, J. Water Infiltration and Water Stability of Compacted Loess Subgrades Based on Vibration Compaction. *Arab. J. Sci. Eng.* **2022**, *47*, 4987–5001.

15. Liu, K.; Ye, W.; Sun, X.; Dang, F.; He, M.; Duan, Z.; Wang, Q. Revealing the mechanisms of water permeability enhancement of urban loess subgrades due to vibration application. *Transp. Geotech.* **2024**, *48*, 101346.
16. Zhang, J.; Hu, H.; Peng, J.; Zhang, Y.; Zhang, A. Enhanced understanding of subgrade soil hydraulic characteristics: Effects of wetting–drying cycles and stress states on subgrade water migration. *J. Hydrol.* **2024**, *635*, 131165.
17. Yue, J.; Zhang, H.; Zhang, Y.; Xu, S. Experimental investigation of influence of amide polymer on loess for subgrade. *Sci. Rep.* **2024**, *14*, 12229.
18. Sim, M.L.; Ling, F.N.L.; Zaihasra, A.T. Analytical study on effect of groundwater table and rainfall infiltration on the stability and performance of Muar Trial Subgrade by using PLAXIS 2D. In Proceedings of the IOP Conference Series: Earth and Environmental Science, Bangkok, Thailand, 22–24 May 2019; IOP Publishing: Bristol, UK, 2024; Volume 1347, p. 012061.
19. Liu, L.; Li, Z.; Cai, G.; Liu, X.; Yan, S. Humidity field characteristics in road subgrade constructed with recycled construction wastes. *J. Clean. Prod.* **2020**, *259*, 120977. [CrossRef]
20. Liu, W.; Yu, W.; Fortier, R.; Chen, L.; Lu, Y.; Zhang, M.; Hu, D. Thermal effect of rainwater infiltration into a replicated road subgrade in a cold environmental chamber. *Cold Reg. Sci. Technol.* **2019**, *159*, 47–57.
21. Tai, B.; Yue, Z.; Qi, S.; Wang, P. Experimental and numerical investigation on thermal-moisture-mechanical behaviors on a new anti-frost cutting bed of high-speed railway in deep seasonally frozen ground regions under extreme climate. *Comput. Geotech.* **2021**, *136*, 104251.
22. Chen, S.L.; Dong, K.H.; Shi, J.J.; Yu, T.; Huang, J. Infiltration Time Effects of Compressibility of Cement Soil. *Appl. Mech. Mater.* **2013**, *405*, 10–14.
23. Zhao, L.; Gray, D.M.; Toth, B. Influence of soil texture on snowmelt infiltration into frozen soils. *Can. J. Soil. Sci.* **2002**, *82*, 75–83. [CrossRef]
24. Stuurop, J.C.; van der Zee, S.E.; French, H.K. The influence of soil texture and environmental conditions on frozen soil infiltration: A numerical investigation. *Cold Reg. Sci. Technol.* **2022**, *194*, 103456.
25. Wen, P.; Wang, C.; Gao, X.; Chen, M.; Chai, Z. Effect of environmental factors on mechanical properties and damage characteristics of mine overburden used in subgrade. *Int. J. Pavement Eng.* **2023**, *24*, 2032698.
26. Li, W.; Yu, L.; Tan, Y.; Wu, L.; Qian, J. Mechanical properties and impact behavior of frozen clay: Insights from static mechanical tests, fly-plate tests, and split-Hopkinson pressure bar analysis. *Phys. Fluids* **2024**, *36*, 057138.
27. Peng, Y.; Yu, L.; Qian, J.; Li, W.; Zhang, T.; Zhou, L. Dynamic tensile behavior and crack propagation in coral aggregate seawater shotcrete: Experimental investigation and numerical simulation. *Cem. Concr. Compos.* **2025**, *159*, 106010.

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Article

Development of Enhanced Stress Prediction Models for Fixed Traffic Loads on Flexible Pavements Based on Response Surface Methodology (RSM) and Machine Learning (ML) Techniques

Adham Mohammed Alnadish ¹, Madhusudhan Bangalore Ramu ^{2,*}, Abdullah O. Baarimah ² and Aawag Mohsen Alawag ³

¹ Department of Civil Engineering, National University of Sciences and Technology (NUST), Balochistan Campus, Quetta 87300, Pakistan; adhmalnadish@gmail.com

² Department of Civil and Construction Engineering, College of Engineering, A'Sharqiyah University, Ibra 400, Oman; abdullah.baarimah@asu.edu.om

³ Faculty of Civil Engineering, Universiti Teknologi MARA (UiTM), Shah Alam 40450, Selangor, Malaysia; alawaj711mohsen@gmail.com

* Correspondence: madhusudhan.ramu@asu.edu.om

Abstract: Pavement design is influenced by traffic load, which affects its lifespan. Traditional methods classify traffic load into fixed traffic, fixed vehicle, variable traffic, and vehicle/axle loads. In fixed traffic, pavement thickness is based on the maximum expected wheel load without considering load repetitions. Meanwhile, in fixed vehicle scenarios, it is calculated by the repetitions of a standard axle load. For nonstandard axle loads, the equivalent axle load is determined by multiplying repetitions by the corresponding equivalent load factor. In variable traffic, each axle and its repetitions are analyzed independently. This study proposes enhanced models for fixed traffic loads, focusing on single, dual, and tridem axles in a single-layer pavement model, to improve stress prediction accuracy. The results show that a quadratic model with a base-10 logarithmic transformation accurately predicts stresses. Additionally, machine learning models, especially Gradient Boosting, provided more accurate predictions than traditional models, with lower mean squared error (MSE) and root mean squared error (RMSE). The results show that these models are effective in predicting the stress in pavement. These findings provide valuable insights that can lead to better pavement design and more effective maintenance practices.

Keywords: pavement; models; fixed traffic; RSM; base-10 logarithmic transformation; quadratic model; machine learning; gradient boosting

1. Introduction

There are several major factors that lead to the deterioration of asphalt pavement, and traffic loads are among the most important. These loads significantly affect pavement performance due to their repetition over time [1,2]. Traffic loads, their frequency, and the resulting stresses are pivotal in determining the performance and longevity of the pavement. When vehicles pass, they generate different stresses that can cause various types of damage, depending on the type and weight of the load. As the weight of the load increases, the likelihood of pavement damage also increases, leading to deformation, cracking, and fatigue.

Pavements are designed to withstand a certain number of load repetitions throughout their service life; however, excessive repetition of heavy loads can exceed the structural capacity of the pavement [3–5]. Each time a vehicle passes over the pavement, it generates

stress that accumulates over time, which can lead to fatigue failure, especially in areas with heavy traffic. Moreover, determining the stresses generated by vehicles in pavements poses a significant challenge for designers. This process involves understanding how different types of vehicles, loads, and traffic patterns affect pavement performance. Therefore, understanding and addressing these factors is essential for maintaining the safety and longevity of asphalt pavements [6–8].

The impact of traffic and vehicles on pavement design can be classified into three main categories: fixed traffic, fixed vehicle, and variable vehicle and traffic. In fixed traffic scenarios, pavement thickness is determined based on the load of a single wheel, disregarding the number of load repetitions [9,10]. However, in situations involving different types of wheels, such as heavy wheel loads on highway pavements, they need to be converted to an equivalent single-wheel load (ESWL). Conversely, the fixed vehicle method considers the number of load repetitions of a standard axle, typically a single axle with dual tires and a load of 18 kips (80 kN). For pavements exposed to various axle types, like tandem or tridem axles, they are converted to an equivalent axle load factor (EALF) to achieve an equivalent effect based on an 18 kip (80 kN) single-axle load [11,12].

The variable traffic and vehicles method treats vehicles and traffic separately, eliminating the need to assign equivalent factors for different axles. This method evaluates strains and deflections for each load individually. Several methods have been developed to determine the equivalent single-wheel load (ESWL), including the equal vertical stress criterion, equal vertical deflection criterion, equal tensile strain criterion, criterion based on equal contact pressure, and criterion based on equivalent contact radius. The semi-rational equal vertical stress criterion, introduced by Boyd and Foster [10,13], calculates the produced stress (ESWL) based on pavement layer thickness.

However, safety concerns arose from accelerated traffic tests, leading to the equal vertical deflection criterion by Foster and Ahlvin [10,14]. This method assumes a homogeneous half-space pavement layer and determines the developed vertical deflection at the top of the subgrade layer using Boussinesq solutions. Huang, 1969 [15], suggested that this method still poses safety risks, as some pavement thicknesses exceed those obtained by the Boyd and Foster method. Other methods, such as the criterion based on equal contact pressure and the criterion based on equivalent contact radius, have been developed to address these challenges.

Previous methods are ineffective in explaining the stress concentration generated by tire contact overlap in tandem and tridem axles, resulting in an incomplete understanding of stress distribution. However, the thickness of pavement layers plays a crucial role in load distribution; thicker layers help minimize stress on underlying layers and enhance the ability to support loads. Likewise, axle configuration, in terms of tire spacing, has a significant effect on how stress is distributed. Greater spacing between tires minimizes overlap stresses, while closely spaced tires increase stress overlap [16–19]. These challenges show the need for advanced modeling tools like PITRA PAVE.

PITRA PAVE is a tool used to study pavement performance by analyzing how it responds to structural stresses induced by traffic loads. It contains a wide variety of materials, including asphalt mixtures, granular layers, and subgrade. The tool models multiple load configurations, including single, dual, and multi-axle loads, while accounting for dynamic loading conditions. Furthermore, PITRA PAVE enables for wide assessments of stress, strain, and deflection within pavement layers. This provides important information on the long-term performance of pavement infrastructure. Moreover, PITRAPAVE offers advanced functions, including dynamic loading simulations and better material analysis. These advancements enable for a more thorough understanding of pavement behavior under complex loading situations, surpassing earlier tools like KENPAVE [20–22].

In addition to PITRA PAVE, response surface methodology (RSM) is widely used to model the performance of pavements. RSM applies statistical tools to investigate correlations between variables like layer thickness and tire spacing, predicting pavement behavior under load [23,24]. RSM provides valuable insights, however, it has its limitations. It struggles to capture complex, nonlinear interactions and manage the large datasets common in real-world pavement systems [25,26]. This is where machine learning (ML) starts to assist the development of models. Unlike RSM, which relies on predetermined mathematical assumptions, ML algorithms can evaluate enormous datasets, find nonlinear patterns, and generate predictions without simplifications. Integrating ML into the modeling process provides for more precise and trustworthy pavement performance forecasts, leading to better assessments of long-term pavement behavior [27,28].

This research aims to develop models to improve stress prediction in flexible pavements under fixed traffic conditions. A homogeneous single-layer model was adopted to isolate the effects of axle configuration (single, tandem, tridem), tire spacing, and pavement thickness, avoiding the complexity of multi-layer systems. Response surface methodology (RSM) has been used to model stresses from single, tandem, and tridem axles. To further enhance the accuracy of these models, machine learning (ML) was integrated, enabling more precise predictions by analyzing complex patterns.

2. Methods

In this study, the PITRA PAVE v1.0.0 tool was used to evaluate the stress caused by different axle configurations, including single, tandem, and tridem axles. However, The analysis focused on a homogeneous, linear-elastic, isotropic single-layer pavement model with a layer thickness ranging from 0 to 50 inches in 5-inch increments. The tire load was set at 4500 lb, and the tire pressure was maintained at 100 psi, while the elastic modulus and Poisson's ratio were kept constant at 10,000 psi and 0.5, respectively. According to the outputs from PITRA PAVE, vertical stress is primarily influenced by pavement thickness rather than by the elastic modulus or Poisson's ratio. However, the tire pressure was fixed at 100 psi, as variations minimally affect vertical stress, and load was set at 4500 lb because vertical stress scales linearly with load. Stresses were measured at multiple locations, as illustrated in Figure 1, to identify the critical stress point and adopt it for the analysis and model development. For the single axle, tandem axle, and tridem axles, the tire spacings in the Y direction were set to 7, 14, and 21 inches, respectively. In the X direction, spacings for the tandem and tridem axles were 24, 48, and 72 inches. Subsequently, Design-Expert V.13 was employed to develop prediction models for the stresses produced by the different axle types. The models considered pavement thickness and tire spacing in both the X and Y directions, with the response variable being the developed critical stresses.

Machine Learning Approach for Stress Prediction Modeling

In this study, a machine learning approach was employed to optimize and assess several regression models for predicting stress values in single, tandem, and tridem axles. The procedure began by loading the dataset and applying preprocessing steps, such as a log transformation to the target variable (stress) to stabilize its variance. The dataset was then divided into training and testing subsets, with 80% used for training and 20% for testing and evaluation. Various regression models were trained, including linear regression, ridge, lasso, support vector regression (SVR), decision tree, random forest, gradient boosting, LightGBM, and CATBoost. To improve model performance, polynomial features were added. The models were evaluated using performance metrics like mean squared error (MSE), R^2 , mean absolute error (MAE), and root mean squared error (RMSE). The best-performing model was chosen based on these metrics. Additionally, to assess model

interpretability and feature significance, diagnostics such as residual plots and Shapley Additive Explanations (SHAP) analysis were performed. The results were visualized to provide a better understanding of the model's performance. A flowchart summarizing the entire process is shown in Figure 2.

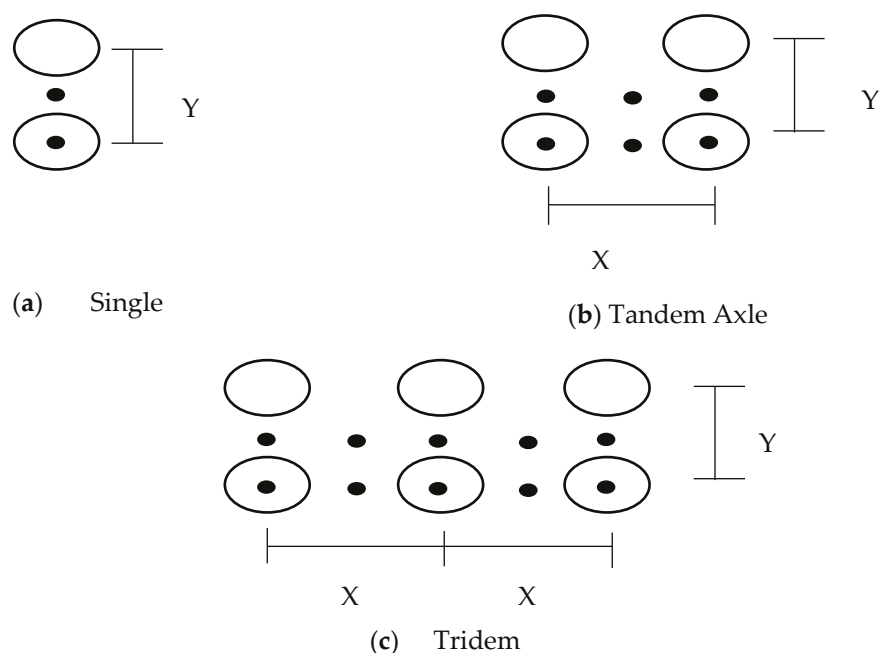


Figure 1. Locations of stress measurements.

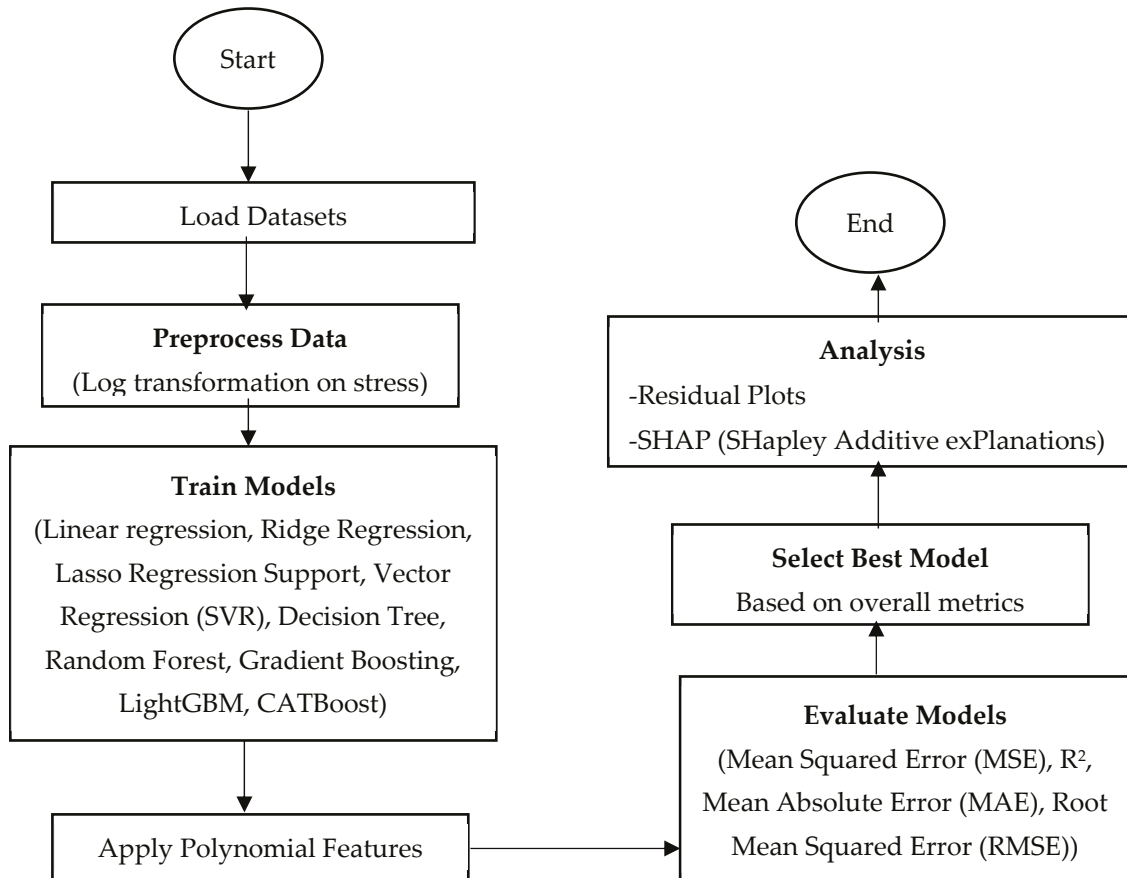


Figure 2. Flow chart of machine learning approach.

3. Analysis and Results

3.1. RSM Results for Single, Tandem, and Tridem Axles

This section analyzes the data produced by response surface methodology (RSM) for several axle configurations, including single, tandem, and tridem.

3.1.1. Single Axle with Dual Tires

Table 1 provides a comprehensive summary of the outputs generated by the PITRA PAVE v1.0.0 software. The data indicate that, as the depth of the assessment points increases, the values of the stresses exhibit a significant decline. This trend highlights the relationship between depth and stress distribution within the pavement structure, emphasizing the importance of considering depth in the design and analysis of pavement systems. Moreover, the analysis reveals that reduced spacing between tires correlates with increased stress levels. This phenomenon can be attributed to stress overlap, where closely spaced tires distribute their load over a smaller area, resulting in higher localized stress concentrations. Understanding this relationship is crucial for optimizing tire configurations to minimize pavement damage. Further, to investigate the interactions between tire spacing (in the Y direction), pavement thickness, and the resultant stress response, a statistical evaluation was conducted using response surface methodology (RSM). This approach allowed for the development of a predictive model that assesses stress at various depths within the pavement layer. Importantly, a base 10 logarithmic transformation was applied to the data to enhance the model's performance, as preliminary analyses indicated that the untransformed model did not yield satisfactory results. Table 2 presents the fit summary of the various models generated during this analysis. Notably, the quadratic model emerged as the most effective predictor of stress, supported by its significant p -value and superior adjusted and predicted R^2 values when compared to alternative models. In contrast, the cubic model was found to be aliased, suggesting that it may not provide reliable predictions within the studied parameters.

Table 1. Outputs of PITRA PAVE for single axle with dual tires.

Run	Y Spacing (A) (in)	Thickness (B) (in)	Stress (psi)	Run	Y Spacing (A) (in)	Thickness (B) (in)	Stress (psi)
1	14	0	100.03	18	7	30	4.5288
2	14	5	49.749	19	7	35	3.3682
3	14	10	19.733	20	7	40	2.5941
4	14	15	11.376	21	7	45	2.0524
5	14	20	7.8512	22	7	50	1.6592
6	14	25	5.5819	23	21	0	100.24
7	14	30	4.1164	24	21	5	49.372
8	14	35	3.1359	25	21	10	18.528
9	14	40	2.4537	26	21	15	9.4982
10	14	45	1.9623	27	21	20	5.9935
11	14	50	1.5986	28	21	25	4.5274
12	7	0	178.14	29	21	30	3.5376
13	7	5	61.325	30	21	35	2.8012
14	7	10	29.161	31	21	40	2.2553
15	7	15	15.742	32	21	45	1.8459
16	7	20	9.5875	33	21	50	1.534
17	7	25	6.3817				

Table 2. Fit summary of the various models for single axle with dual tires.

Fit Summary				
Source	Sequential <i>p</i> -Value	Adjusted R ²	Predicted R ²	Performance
Linear	<0.0001	0.9197	0.9047	
2FI	0.4196	0.9188	0.892	
Quadratic	<0.0001	0.9903	0.9862	Suggested
Cubic	<0.0001	0.9973	0.9953	Aliased

Table 3 presents the ANOVA results for the quadratic model. The analysis reveals that the impact of tire spacing alone on the model is insignificant, as indicated by a *p*-value greater than 0.05. Conversely, the effect of pavement thickness is significant, with a *p*-value below 0.05, showing its importance in stress response. Additionally, the interaction between pavement thickness and tire spacing was found to be significant. This is because, at depths close to the surface, there is no overlap between stresses but, as the location of analysis goes deeper, there will be an overlap between the developed stresses from the tires. Table 3 also summarizes the model's fit statistics. Notably, the difference between the adjusted and predicted R² values is less than 2, indicating strong agreement between the predicted and observed values. The adequate precision value of 76, which far exceeds the threshold of 4, further suggests that the model is not only effective but also capable of reliably navigating the design space, making it a valuable tool for engineers in pavement design. Figure 3 illustrates several important diagnostic plots for the quadratic model: the normal probability plot, residuals versus predicted values, residuals versus run, and predicted versus actual values. The normal probability plot is particularly useful for assessing the distribution of residuals; a straight line on this plot indicates a favorable fit for linear regression. Any deviations from this line, such as right or left skewness, could suggest potential issues with the model's assumptions about error distribution [29,30]. To assess homoscedasticity, the assumption that the variance of errors is constant across all levels of the independent variables, externally studentized residuals were plotted against predicted values. A scatter plot with symmetrically shaped errors around zero supports the assumption of homoscedasticity, whereas any asymmetric patterns would indicate heteroscedasticity [31]. Figure 3 demonstrates a symmetrical pattern around zero, confirming the model's suitability for accurately predicting the developed stresses. The residuals versus run number plot is instrumental in detecting autocorrelation among disturbances. The absence of autocorrelation is indicated by residuals transitioning randomly between positive and negative values, as observed in Figure 3. This randomness suggests that the stresses are well modeled by the independent variables [32]. However, the actual versus predicted values plot provides a visual representation of the model's predictive capability. Figure 3 shows a strong correlation between actual and predicted values, with data points closely aligning with the 1:1 line. This alignment underscores the model's accuracy in predicting developed stresses under various loading conditions, validating its utility in pavement design. Equation (1) describes the quadratic model.

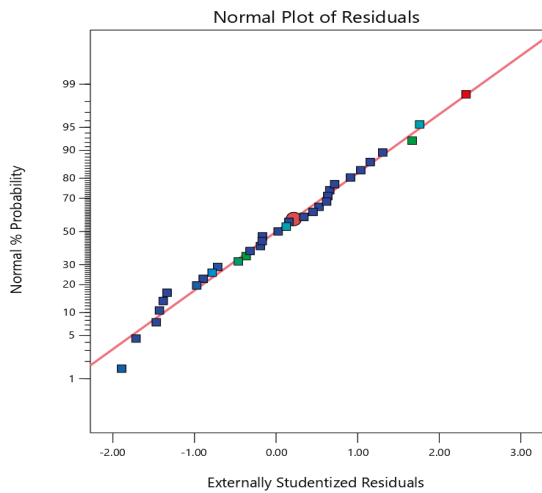
$$\sigma_z = 10^{(2.31928 - 0.026869*A - 0.072919*B + 0.000267*A*B + 0.000387*A^2 + 0.000676*B^2)} * \left(\frac{\text{tire load}}{4500} \right) \quad (1)$$

where:

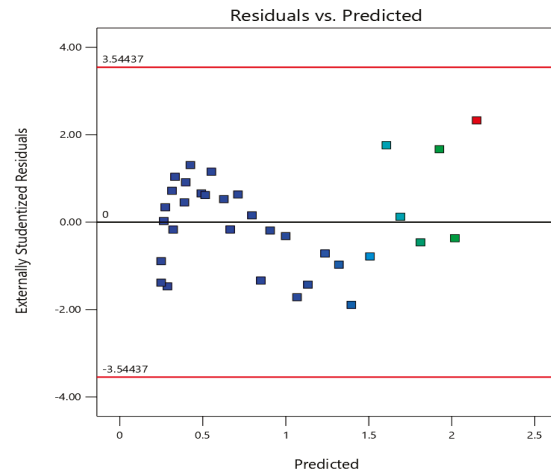
σ_z = Vertical stress;

A = spacing between tires in the Y direction;

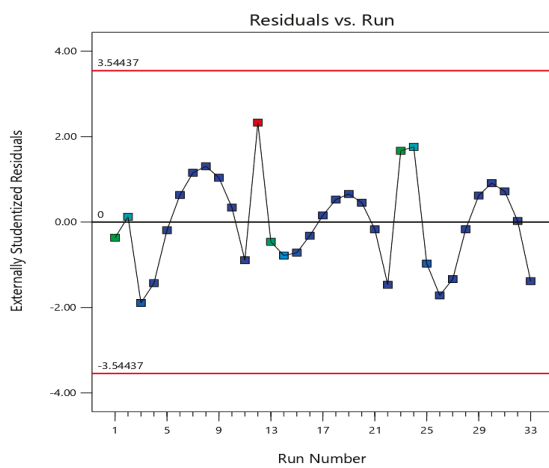
B = thickness of the pavement.



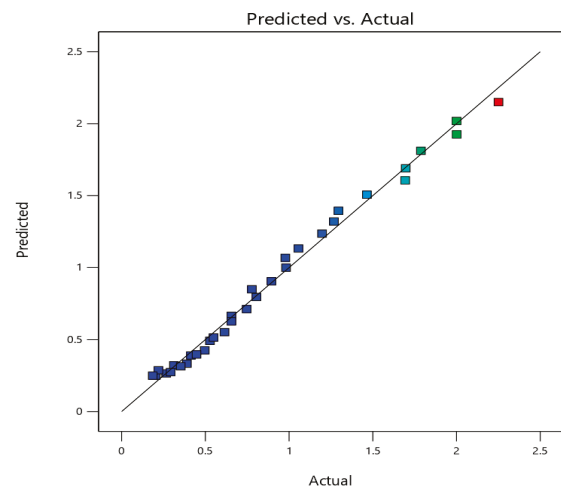
(a) Normal probability



(b) externally studentized residuals versus predicted values



(c) Residuals versus run number plot



(d) Actual versus predicted values plot

Figure 3. Plots analyzing the quadratic model of developed stress for single axle.**Table 3.** ANOVA for quadratic model for single axle model.

Source	Sum of Squares	df	Mean Square	F-Value	p-Value	Performance
Model	11.19	5	2.24	651.13	<0.0001	significant
A-A	0	1	0	0.0035	0.9534	
B-B	1.61	1	1.61	469.74	<0.0001	
AB	0.0192	1	0.0192	5.58	0.0256	
A ²	0.0026	1	0.0026	0.7674	0.3887	
B ²	0.7346	1	0.7346	213.75	<0.0001	
Residual	0.0928	27	0.0034			
Cor Total	11.28	32				
Fit statistics						
Std. Dev.	0.0586		R ²			0.9918
Mean	0.8933		Adjusted R ²			0.9903
C.V. %	6.56		Predicted R ²			0.9862
			Adeq Precision			76.0539

3.1.2. Tandem Axle with Dual Tires

The stresses induced by a tandem axle with dual tires were evaluated using the PITRA PAVE software. As shown in Table 4, the resulting stress values exhibit a clear decrease as the measurement depth increases. Additionally, the spacing between tires in both the Y and X directions plays a significant role in stress development. Specifically, reduced spacing tends to result in higher stress levels within a certain range. To better understand this relationship, a statistical analysis was performed using response surface methodology (RSM), focusing on the depth at which the stress was measured and the tire spacing. This method enabled the construction of a predictive model that effectively captures the complex dynamics of stress distribution. The use of a base 10 logarithmic transformation further enhanced the model's accuracy by stabilizing variance and improving the overall fit. The fit summary provided in Table 5 suggests the use of a quadratic model, while the cubic model was excluded due to aliasing issues. The quadratic model was chosen because of its significant p -value and high adjusted R^2 value, both of which indicate a strong correlation between the key influencing factors and the response variable. The detailed ANOVA results for the quadratic model are shown in Table 6, underscoring the significant impact of both tire spacing and stress measurement depth on stress development, as evidenced by consistently low p -values below 0.05. Moreover, the fit statistics summarized in Table 6 reveal only a minimal difference between the adjusted and predicted R^2 values, which further confirms the model's reliability. The adequate precision value, exceeding 4, reinforces the model's robustness, making it a reliable tool for practical applications in pavement design.

$$\sigma_z = 10^{(2.37052 - 0.067929*A - 0.003682*B - 0.026319*C - 0.000087*A*B + 0.000267*A*C - 0.0000025*B*C + 0.000706*A^2 + 0.000043*B^2 + 0.000379*C^2)} * \left(\frac{\text{tire load}}{4500} \right) \quad (2)$$

where:

σ_z = Vertical stress;

A = thickness of the pavement;

B = spacing between tires in the X direction;

C = spacing between tires in the Y direction.

Figure 4 shows the residual plots for the quadratic model, including the normal probability plot, residuals versus predicted values, residuals versus run, and actual versus predicted values. These plots are crucial for evaluating the model's performance through the assumptions. The normal probability plot is specifically used to assess the distribution of the residuals. As seen in Figure 4, the residuals are close to a straight line, indicating that the errors are normally distributed. This alignment suggests that the quadratic model is well suited for the data. Homoscedasticity, which refers to the assumption of equal variance across the range of predicted values, is examined through the plot of externally studentized residuals versus predicted values. As depicted in Figure 4, the residuals are symmetrically distributed around zero, with no apparent pattern or systematic deviation. This symmetry indicates that the model's predictions are consistent across the range of data. Autocorrelation can undermine the model's effectiveness by indicating that the residuals are not independent, which is another key assumption for regression models. In Figure 4, the residuals appear to transition randomly between positive and negative values, with no discernible pattern. This randomness suggests that there is no autocorrelation, meaning the stresses are well captured by the independent variables in the model. Finally, the plot of actual versus predicted values demonstrates the model's predictive accuracy. As shown in Figure 3, the data points closely align along the 1:1 line, indicating a strong correlation between the actual values and the values predicted. The strong agreement between actual and predicted values indicates the model's ability to predict accurately.

Table 4. Outputs of PITRA PAVE for tandem axle with dual tires.

Run	Thickness (A) (in)	X Spacing (B) (in)	Y Spacing (C) (in)	Stress (psi)	Run	Thickness (A) (in)	X Spacing (B) (in)	Y Spacing (C) (in)	Stress (psi)
1	0	48	14	100.02	51	30	72	21	3.5763
2	5	48	14	49.751	52	35	72	21	2.8552
3	10	48	14	19.747	53	40	72	21	2.3253
4	15	48	14	11.42	54	45	72	21	1.9317
5	20	48	14	7.9388	55	50	72	21	1.6345
6	25	48	14	5.7221	56	0	48	21	100.24
7	30	48	14	4.3103	57	5	48	21	49.374
8	35	48	14	3.3781	58	10	48	21	18.541
9	40	48	14	2.735	59	15	48	21	9.5364
10	45	48	14	2.2721	60	20	48	21	6.0706
11	50	48	14	1.9268	61	25	48	21	4.6606
12	0	24	14	99.902	62	30	48	21	3.7227
13	5	24	14	49.798	63	35	48	21	3.0333
14	10	24	14	20.024	64	40	48	21	2.526
15	15	24	14	12.088	65	45	48	21	2.1453
16	20	24	14	8.8936	66	50	48	21	1.8521
17	25	24	14	6.8103	67	0	48	7	178.09
18	30	24	14	5.4004	68	5	48	7	61.327
19	35	24	14	4.3897	69	10	48	7	29.176
20	40	24	14	3.6312	70	15	48	7	15.788
21	45	24	14	3.0442	71	20	48	7	9.6781
22	50	24	14	2.5809	72	25	48	7	6.5263
23	0	72	14	100.08	73	30	48	7	4.7283
24	5	72	14	49.75	74	35	48	7	3.6167
25	10	72	14	19.735	75	40	48	7	2.8821
26	15	72	14	11.383	76	45	48	7	2.3688
27	20	72	14	7.8658	77	50	48	7	1.9938
28	25	72	14	5.6076	78	0	24	7	177.72
29	30	72	14	4.156	79	5	24	7	61.387
30	35	72	14	3.1912	80	10	24	7	29.525
31	40	72	14	2.5253	81	15	24	7	16.538
32	45	72	14	2.0499	82	20	24	7	10.729
33	50	72	14	1.7011	83	25	24	7	7.7046
34	0	72	7	170.46	84	30	24	7	5.8928
35	5	72	7	61.326	85	35	24	7	4.6858
36	10	72	7	29.163	86	40	24	7	3.8213
37	15	72	7	15.749	87	45	24	7	3.1724
38	20	72	7	9.6023	88	50	24	7	2.6707
39	25	72	7	6.4078	89	0	24	21	100.01
40	30	72	7	4.569	90	5	24	21	49.413
41	35	72	7	3.4242	91	10	24	21	18.772
42	40	72	7	2.6667	92	15	24	21	10.048
43	45	72	7	2.1411	93	20	24	21	6.8141
44	50	72	7	1.7629	94	25	24	21	5.618
45	0	72	21	100.3	95	30	24	21	4.702
46	5	72	21	49.373	96	35	24	21	3.9576
47	10	72	21	18.53	97	40	24	21	3.3561
48	15	72	21	9.5044	98	45	24	21	2.8682
49	20	72	21	6.007	99	50	24	21	2.47
50	25	72	21	4.5524					

Table 5. Fit summary of the various models for tandem axle with dual tires.

Source	Sequential <i>p</i> -Value	Adjusted R ²	Predicted R ²	Performance
Linear	<0.0001	0.9027	0.8962	Suggested Aliased
2FI	0.2316	0.904	0.8904	
Quadratic	<0.0001	0.9887	0.9869	
Cubic	<0.0001	0.9971	0.9962	

Table 6. ANOVA for quadratic model for tandem axle model.

Source	Sum of Squares	df	Mean Square	F-Value	p-Value	Performance
Model	30.07	9	3.34	951.31	<0.0001	significant
A-A	2.41	1	2.41	684.99	<0.0001	
B-B	0.0547	1	0.0547	15.58	0.0002	
C-C	0.0583	1	0.0583	16.6	0.0001	
AB	0.0726	1	0.0726	20.66	<0.0001	
AC	0.0574	1	0.0574	16.36	0.0001	
BC	7.8×10^{-6}	1	7.8×10^{-6}	0.0022	0.9626	
A ²	2.4	1	2.4	684.32	<0.0001	
B ²	0.0135	1	0.0135	3.85	0.053	
C ²	0.0076	1	0.0076	2.16	0.1451	
Residual	0.3126	89	0.0035			
Cor Total	30.38	98				
Fit statistics						
Std. Dev.	0.0593			R ²	0.9897	
Mean	0.9337			Adjusted R ²	0.9887	
C.V. %	6.35			Predicted R ²	0.9869	
				Adeq Precision	100.3383	

3.1.3. Tridem Axle with Dual Tires

The stress of a tridem axle with dual tires was analyzed using the PITRA PAVE software. As shown in Table 7, with increasing depth, the stresses generated by the loads decrease significantly. The distance between the tires in both the Y and X directions plays a significant role in the stress levels. The smaller the distance, the greater the stress. To identify this relationship, a statistical analysis was conducted using response surface methodology (RSM). This analysis aims to develop a predictive relationship for stress based on loads, pavement depth, and tire spacing. A base 10 logarithmic transformation was applied to enhance model accuracy by stabilizing variance. This is because, without the transformation, the performance of the models was poor. The fit summary in Table 8 shows that the quadratic model is suggested, while the cubic model is rejected. The quadratic model's significant *p*-value and high adjusted R² indicate a strong correlation between tire spacing, depth, and stress response. The ANOVA results listed in Table 9 describe the significant influence of tire spacing and measurement depth on stress development, with *p*-values consistently below 0.05. Additionally, the fit statistics in Table 7 show a minimal difference between the adjusted and predicted R² values. The adequate precision value, exceeding 4, indicates the reliability of the model. Equation (3) shows the quadratic model.

$$\sigma_z = 10^{(2.40058 - 0.064288*A - 0.026072*B - 0.005825*C + 0.000268*A*B - 0.000149*A*C - 0.000013*B*C + 0.000725*A^2 + 0.000386*B^2 + 0.00007*C^2)} * \left(\frac{\text{tire load}}{4500} \right) \quad (3)$$

σ_z = Vertical stress;

A = thickness of the pavement;

B = spacing between tires in the Y direction;

C = spacing between tires in the X direction.

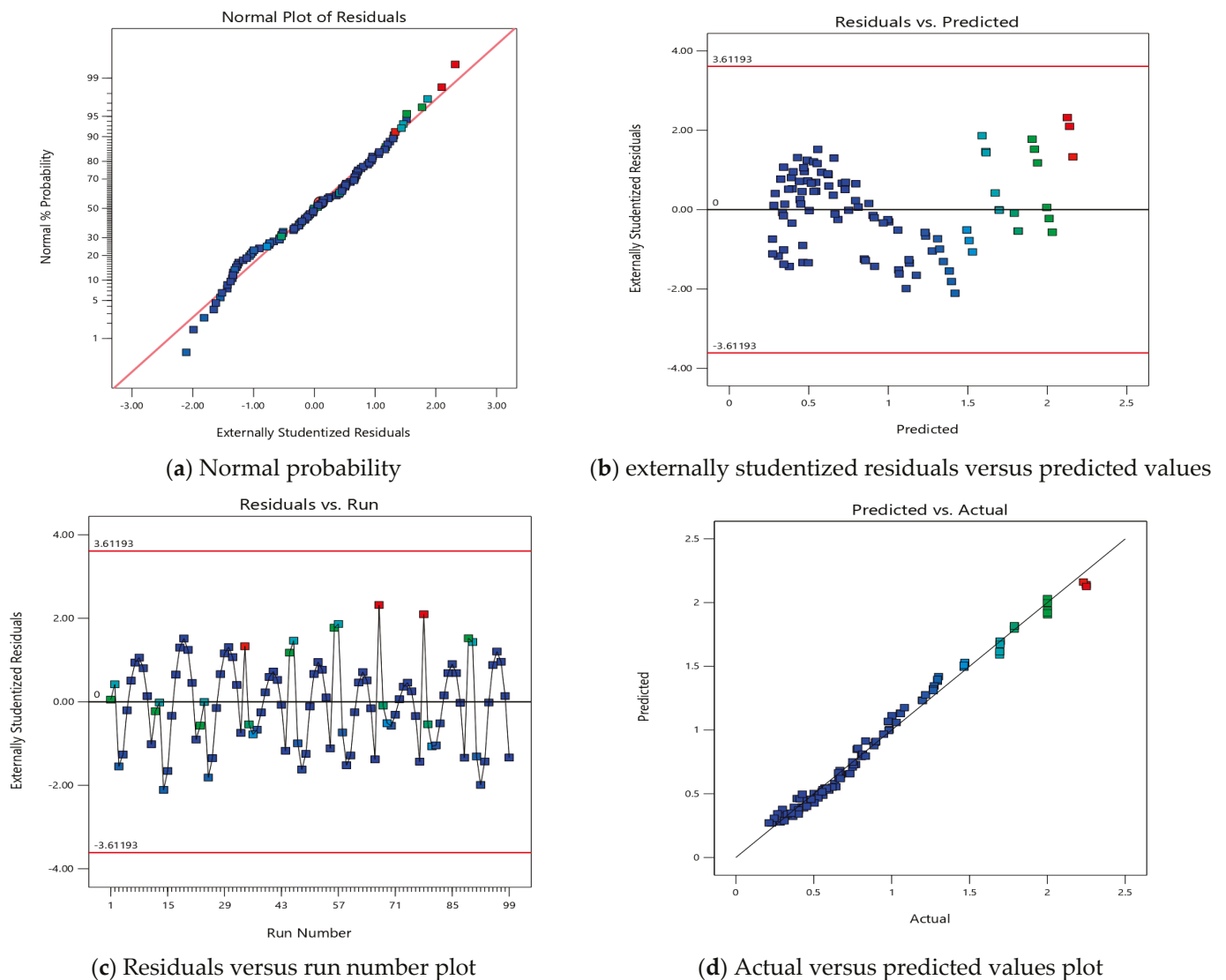


Figure 4. Plots analyzing the quadratic model of developed stress for tandem axle with dual tires.

Figure 5 shows the residual plots of the quadratic model. These plots shown in Figure 5 include the normal probability plot, residuals versus predicted values, residuals versus run number, and actual versus predicted values. However, these plots describe how well the model fits the data and predicts the stress accurately. The normal probability plot is essential for assessing whether the residuals, or errors, follow a normal distribution. The data points on this plot should form a straight line, indicating that the residuals are normally distributed. As seen in Figure 5, the residuals are close to the theoretical line, suggesting that the data are normally distributed. Furthermore, the plot of externally studentized residuals versus predicted values is used to evaluate the reliability of the model. This plot is crucial for assessing homoscedasticity, which refers to the assumption that the error terms (the differences between the observed and predicted values of the dependent variable) have equal variance across all levels of the independent variables. As seen in Figure 5, the residuals are symmetrically distributed around zero, showing no clear pattern or trend. The residuals versus run number plot is used to detect any autocorrelation among the residuals, which would suggest that the errors are not independent. It can be seen in Figure 5 that the residuals transition randomly between positive and negative values, indicating that there is no unsystematic pattern. This randomness suggests that there is no autocorrelation, indicating that the model accurately describes the relationship between

the independent variables and the stresses. On the other hand, the plot of actual versus predicted values illustrates the model's accuracy in predicting the observed values. As noted in Figure 5, the data points closely follow the 1:1 line, indicating a strong correlation between the actual values and those predicted by the model. This close alignment proves the model's effectiveness in predicting the developed stresses.

Table 7. Outputs of PITRA PAVE for tridem axle with dual tires.

Run	Thickness (A) (in)	Y Spacing (B) (in)	X Spacing (C) (in)	Stress (psi)	Run	Thickness (A) (in)	Y Spacing (B) (in)	X Spacing (C) (in)	Stress (psi)
1	0	14	48	100.01	51	30	7	24	7.2569
2	5	14	48	49.753	52	35	7	24	6.0035
3	10	14	48	19.761	53	40	7	24	5.0484
4	15	14	48	11.464	54	45	7	24	4.2924
5	20	14	48	8.0264	55	50	7	24	3.6822
6	25	14	48	5.8623	56	0	7	72	178.28
7	30	14	48	4.5041	57	5	7	72	61.326
8	35	14	48	3.6203	58	10	7	72	29.165
9	40	14	48	3.0163	59	15	7	72	15.756
10	45	14	48	2.582	60	20	7	72	9.6171
11	50	14	48	2.2551	61	25	7	72	6.4339
12	0	14	24	99.892	62	30	7	72	4.6093
13	5	14	24	49.846	63	35	7	72	3.4803
14	10	14	24	20.314	64	40	7	72	2.7392
15	15	14	24	12.19	65	45	7	72	2.2298
16	20	14	24	9.9359	66	50	7	72	1.8666
17	25	14	24	8.0387	67	0	21	48	100.24
18	30	14	24	6.6843	68	5	21	48	49.376
19	35	14	24	5.6434	69	10	21	48	18.553
20	40	14	24	4.8087	70	15	21	48	9.5746
21	45	14	24	4.1261	71	20	21	48	6.1478
22	50	14	24	3.5631	72	25	21	48	4.7939
23	0	14	72	100.13	73	30	21	48	3.9077
24	5	14	72	49.75	74	35	21	48	3.2654
25	10	14	72	19.737	75	40	21	48	2.7967
26	15	14	72	11.39	76	45	21	48	2.4446
27	20	14	72	7.8803	77	50	21	48	2.1702
28	25	14	72	5.6333	78	0	21	24	100.01
29	30	14	72	4.1956	79	5	21	24	49.454
30	35	14	72	3.2464	80	10	21	24	19.016
31	40	14	72	2.5968	81	15	21	24	10.598
32	45	14	72	2.1374	82	20	21	24	7.6348
33	50	14	72	1.8035	83	25	21	24	6.7086
34	0	7	48	178.04	84	30	21	24	5.8664
35	5	7	48	61.329	85	35	21	24	5.1141
36	10	7	48	29.192	86	40	21	24	4.4569
37	15	7	48	15.833	87	45	21	24	3.8905
38	20	7	48	9.7687	88	50	21	24	3.4061
39	25	7	48	6.671	89	0	21	72	100.31
40	30	7	48	4.9277	90	5	21	72	49.373
41	35	7	48	3.8652	91	10	21	72	18.531
42	40	7	48	3.17	92	15	21	72	9.5074
43	45	7	48	2.6853	93	20	21	72	6.0139
44	50	7	48	2.3283	94	25	21	72	4.5659
45	0	7	24	177.67	95	30	21	72	3.5971
46	5	7	24	61.449	96	35	21	72	2.8845
47	10	7	24	29.89	97	40	21	72	2.3635
48	15	7	24	17.333	98	45	21	72	1.9789
49	20	7	24	11.871	99	50	21	72	1.6903
50	25	7	24	9.0275					

Table 8. Fit summary of the various models for tridem axle with dual tires.

Source	Sequential <i>p</i> -Value	Adjusted R ²	Predicted R ²	Performance
Linear	<0.0001	0.883	0.8753	Suggested Aliased
2FI	0.044	0.8893	0.8737	
Quadratic	<0.0001	0.9859	0.9838	
Cubic	<0.0001	0.9964	0.9952	

Table 9. ANOVA for quadratic model for tridem axle model.

Source	Sum of Squares	df	Mean Square	F-Value	p-Value	Performance
Model	27.93	9	3.1	763.87	<0.0001	significant
A-A	2.52	1	2.52	620.06	<0.0001	
B-B	0.0591	1	0.0591	14.54	0.0003	
C-C	0.1624	1	0.1624	39.98	<0.0001	
AB	0.0582	1	0.0582	14.33	0.0003	
AC	0.2107	1	0.2107	51.86	<0.0001	
BC	0.0002	1	0.0002	0.0495	0.8244	
A ²	2.54	1	2.54	624.05	<0.0001	
B ²	0.0079	1	0.0079	1.94	0.1672	
C ²	0.0354	1	0.0354	8.71	0.0041	
Residual	0.3616	89	0.0041			
Cor Total	28.29	98				
Fit statistics						
Std. Dev.	0.0637			R ²	0.9872	
Mean	0.9651			Adjusted R ²	0.9859	
C.V. %	6.6			Predicted R ²	0.9838	
				Adeq Precision	92.953	

3.2. Performance Analysis of Machine Learning Models

This section evaluates the performance of various machine learning models in predicting stress levels for single, tandem, and tridem axles.

3.2.1. Machine Learning Model Performance for Single Axle Stress Prediction

Table 10 displays the performance of different regression models. The results indicate that gradient boosting is the top performer, with the lowest testing MSE (0.006061), RMSE (0.077851), and a high R² value of 0.971168, demonstrating its strong ability to generalize to new data. Likewise, random forest performs well, with low testing MSE (0.016694) and RMSE (0.129204), and a solid R² of 0.920587. However, the decision tree model overfits the training data, obtaining desirable outcomes on the training set (MSE: 0, R²: 1, MAE: 0, RMSE: 0) but struggling with testing data (MSE: 0.020817, R²: 0.900970, MAE: 0.124350, RMSE: 0.144282). The huge disagreement between its training and testing performance indicates it is memorizing the training data instead of learning general patterns. Both linear and ridge regression illustrate uniform results, with ridge slightly beating linear regression on the testing data. On the other hand, lasso regression displays considerable underfitting, with high errors and a negative R² value for testing. LightGBM's performance is insufficient, with large errors in both training and testing. Meanwhile, CatBoost performs moderately. In summary, gradient boosting is the most dependable model. To further confirm the efficiency of gradient boosting, diagnostic plots were studied as shown in Figure 6. The Q-Q plot demonstrated that the residuals follow a normal distribution, verifying the model's correctness. The actual vs. predicted plot also indicated strong alignment, reflecting the model's predictive reliability. Additionally, the externally studentized residuals plot confirmed there was no bias, and the residuals vs. run number plot indicated that the

model's predictions were not influenced by the sequence of the data. These diagnostics together prove gradient boosting as an effective and stable model. Figure 7 presents Shapley Additive Explanations (SHAP) values for the parameters of the thickness and spacing. Thickness (B) has a considerably greater average impact on predictions, which results in noticeable variances in outcomes. Otherwise, Y spacing demonstrates a more consistent and uniform effect, contributing uniformly across different cases.

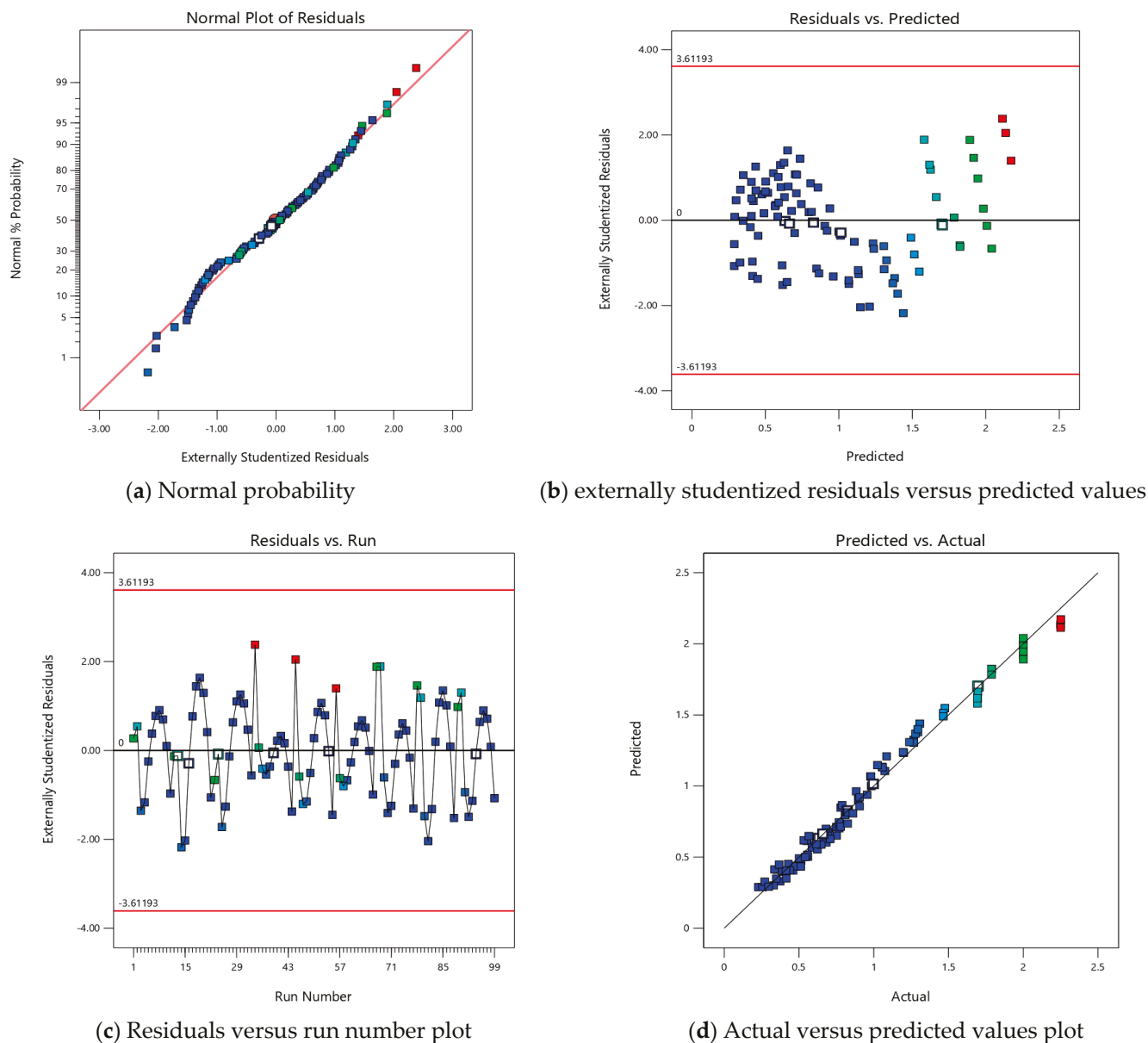


Figure 5. Plots analyzing the quadratic model of developed stress for tridem axle with dual tires.

Table 10. Performance of different regression models for single axle.

Model	Training MSE	Testing MSE	Training R^2	Testing R^2	Training MAE	Testing MAE	Training RMSE	Testing RMSE
Linear Regression	0.016519	0.011680	0.989721	0.944439	0.109154	0.091461	0.128526	0.108072
Ridge Regression	0.019780	0.008936	0.987692	0.957490	0.112288	0.074093	0.140640	0.094531

Table 10. Cont.

Model	Training MSE	Testing MSE	Training R ²	Testing R ²	Training MAE	Testing MAE	Training RMSE	Testing RMSE
Lasso Regression	1.185774	0.741216	0.262126	−2.52603	0.932521	0.789881	1.088932	0.860939
Decision Tree	0.000000	0.020817	1.000000	0.900970	0.000000	0.124350	0.000000	0.144282
Random Forest	0.005351	0.016694	0.996670	0.920587	0.057087	0.089123	0.073151	0.129204
Gradient Boosting	0.000007	0.006061	0.999996	0.971168	0.001996	0.055807	0.002672	0.077851
LightGBM	1.607014	1.075682	0.000000	−4.11712	1.088507	0.930306	1.267680	1.037151
CatBoost	0.000011	0.014862	0.999993	0.929300	0.002677	0.115107	0.003276	0.121910

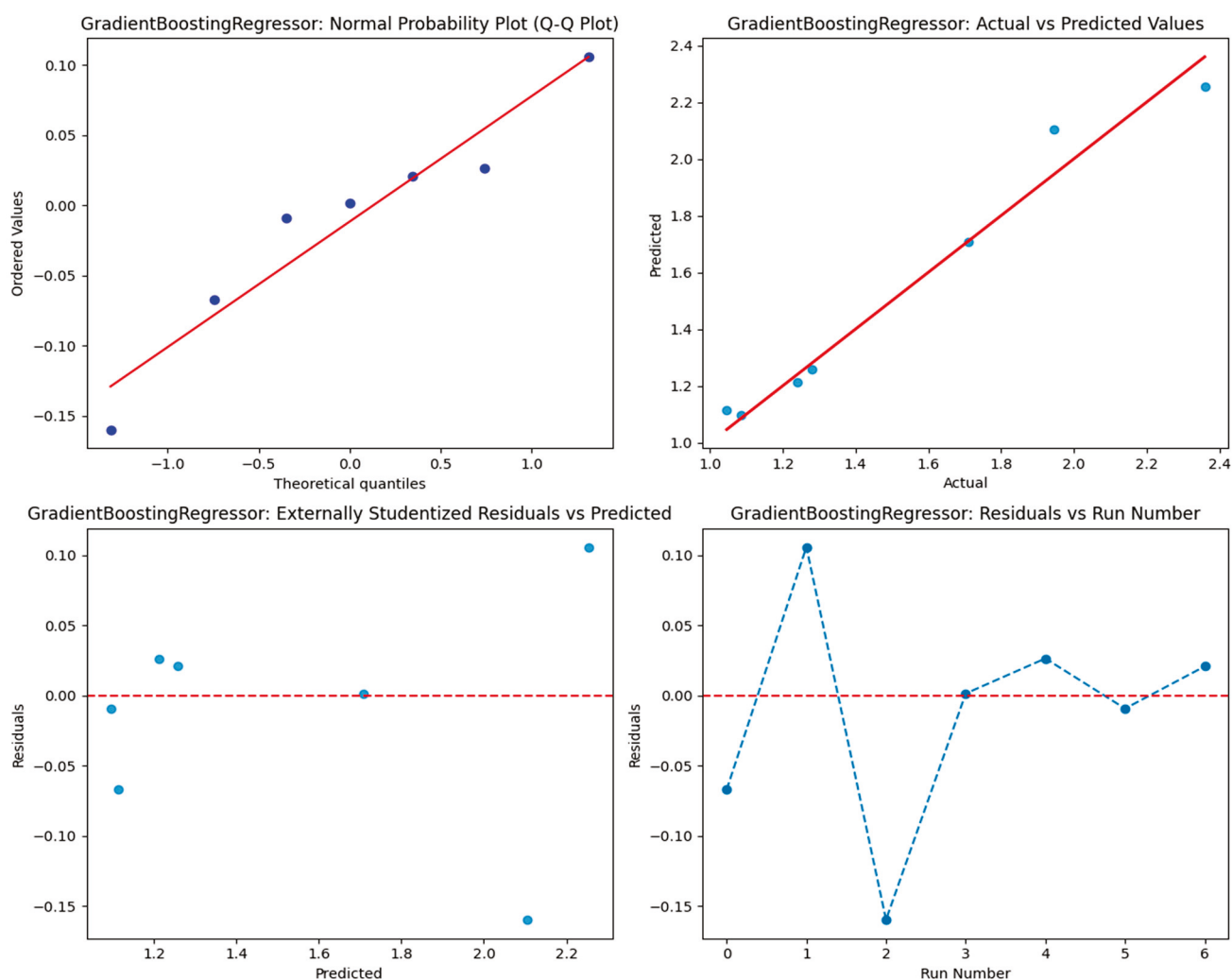


Figure 6. Residuals plots analyzing gradient boosting regression model for single axle.

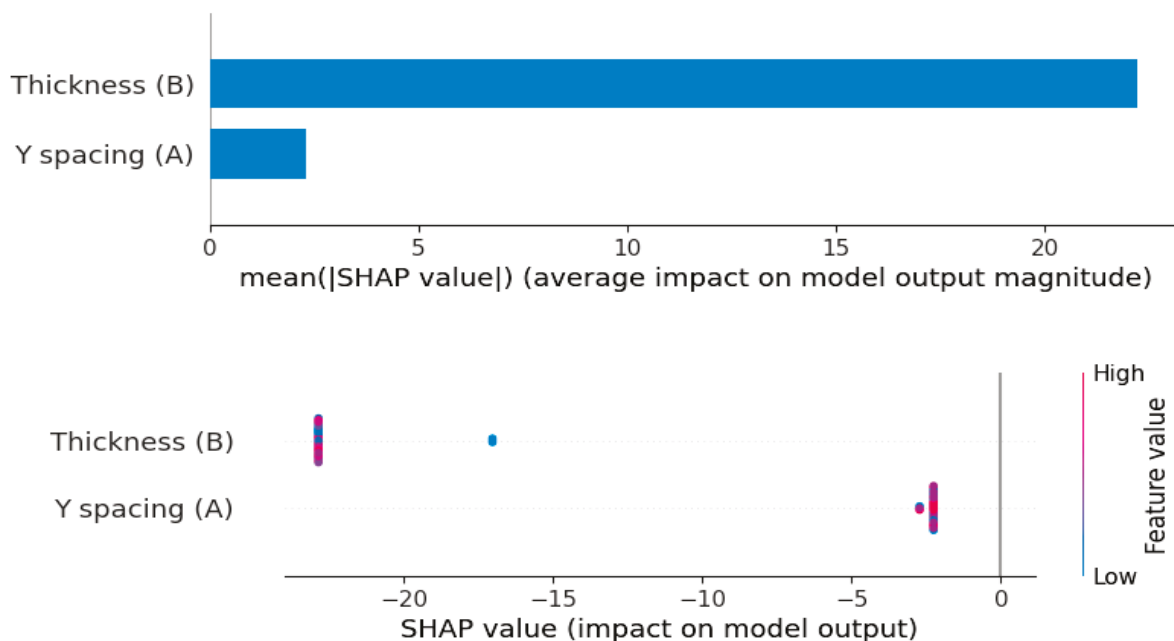


Figure 7. SHAP value analysis for single axle model features.

3.2.2. Machine Learning Model Performance for Tandem Axle Stress Prediction

The results shown in Table 11 illustrate the performance of different regression models, with gradient boosting showing as the best model. It shows the lowest testing MSE (0.001118), MAE (0.023510), and RMSE (0.033436), with a near-perfect R^2 value of 0.999367, demonstrating excellent accuracy and generalization. Random forest likewise works well, with low testing errors (MSE: 0.005714, R^2 : 0.996763) and negligible overfitting. Decision tree, however, displays evidence of overfitting, attaining excellent training results (MSE: 0, R^2 : 1) but inadequate testing performance (MSE: 0.012414, R^2 : 0.992966). Linear and ridge regression produce consistent results with high R^2 values (0.990983 and 0.990604, respectively), although ridge slightly follows linear. Furthermore, lasso regression struggles with severe underfitting, indicated in its large testing errors (MSE: 1.702942, R^2 : 0.035162). LightGBM and CatBoost perform well during training but demonstrate poor generalization ability on testing data. Therefore, gradient boosting is the most dependable model. In addition, the gradient boosting model is further validated by diagnostic plots, as shown in Figure 8. The Q-Q plot indicates that the residuals are regularly distributed, confirming the assumption of normality. The actual vs. predicted values plot illustrates a good correlation between predicted and actual values, presenting high predictive accuracy. The externally studentized residuals vs. predicted values plot indicates no bias, confirming the absence of systematic mistakes in the model. Finally, the residuals vs. run number plot indicates that the model's predictions are independent of the data point sequence, ensuring stability. Overall, gradient boosting stands out as the most dependable model, supported by consistent performance across all evaluation metrics. However, the SHAP value analysis in Figure 9 displays that the thickness of the pavement has the most significant impact on model predictions, as confirmed by its high average SHAP value. Otherwise, X spacing and Y spacing show lesser average affects, with Y spacing demonstrating a more consistent influence.

Table 11. Performance of different regression models for tandem axle.

Model	Training MSE	Testing MSE	Training R ²	Testing R ²	Training MAE	Testing MAE	Training RMSE	Testing RMSE
Linear Regression	0.017564	0.015914	0.985709	0.990983	0.112486	0.100295	0.132530	0.126152
Ridge Regression	0.017797	0.016584	0.985520	0.990604	0.110984	0.101056	0.133406	0.128781
Lasso Regression	1.184310	1.702942	0.036404	0.035162	0.890889	1.113038	1.088260	1.304968
Decision Tree	0.000000	0.012414	1.000000	0.992966	0.000000	0.083175	0.000000	0.111420
Random Forest	0.001753	0.005714	0.998574	0.996763	0.031148	0.059024	0.041867	0.075589
Gradient Boosting	0.000021	0.001118	0.999983	0.999367	0.003692	0.023510	0.004535	0.033436
LightGBM	0.059909	0.098171	0.951256	0.944379	0.179682	0.257142	0.244764	0.313323
CatBoost	0.000005	0.042487	0.999996	0.975928	0.001702	0.111855	0.002162	0.206124

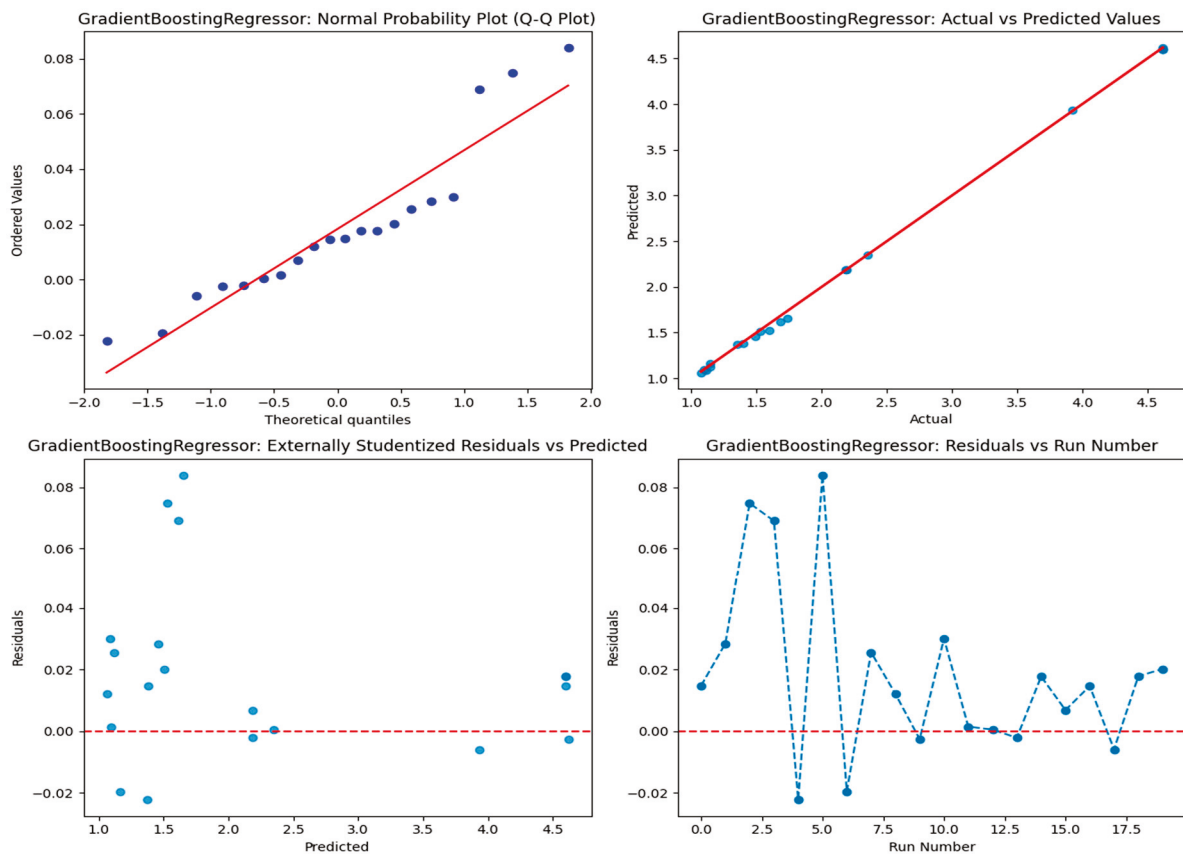


Figure 8. Residuals plots analyzing gradient boosting regression model for tandem axle.

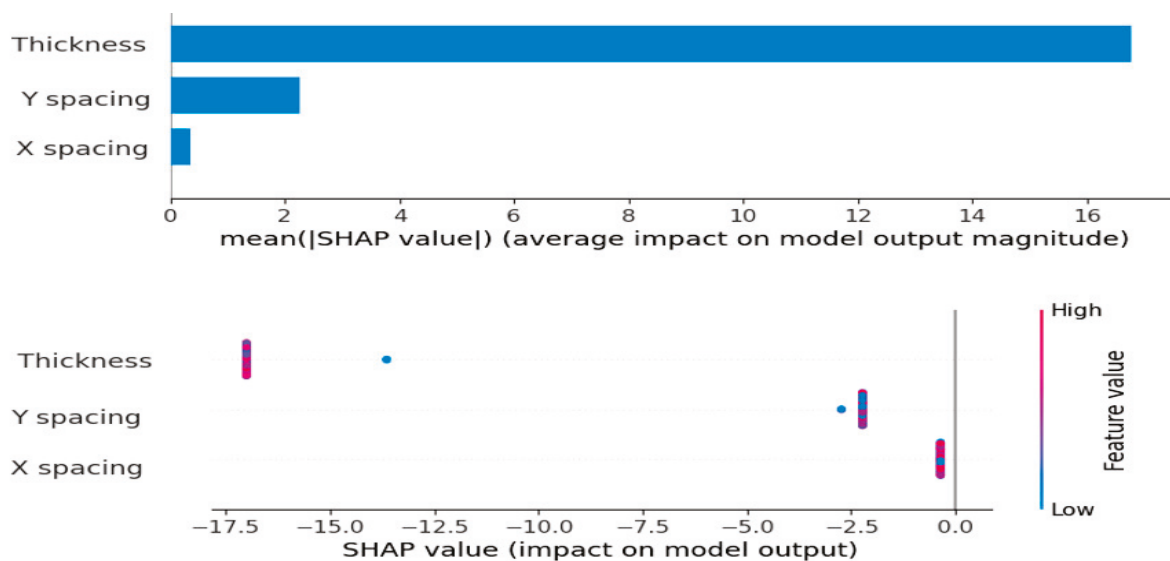


Figure 9. SHAP value analysis for tandem axle model features.

3.2.3. Machine Learning Model Performance for Tridem Axle Stress Prediction

The Table 12 demonstrates the performance of various regression models based on significant metrics. Gradient boosting is demonstrated as the best performer, with the lowest testing MSE (0.003260), MAE (0.037326), and RMSE (0.057093), as well as an almost perfect R^2 value of 0.998295, suggesting superior precision and generalization. Random forest also performs well, with a modest variation between training and testing results (MSE: 0.001837 vs. 0.013046, R^2 : 0.998324 vs. 0.993176), showing low overfitting. On the other hand, decision tree exhibits obvious overfitting, obtaining perfect training results (MSE: 0, R^2 : 1) but poorer testing performance (MSE: 0.014616, R^2 : 0.992354). Both linear regression and ridge regression perform consistently, with high R^2 values and low errors, albeit ridge marginally trails behind linear regression. Lasso regression faces significant challenges with severe underfitting, as shown by its high testing error (MSE: 1.913,948) and a negative R^2 value (-0.001202). While LightGBM and CatBoost perform well during training, their higher testing errors (MSE: 0.114631 and 0.115462, respectively) and lower R^2 values (0.940036 and 0.939601) suggest they struggle to generalize effectively to unseen data. To further test the efficiency of gradient boosting, diagnostic plots were employed to verify its accuracy, as shown in Figure 10. The normal probability plot (Q-Q Plot) demonstrates that the residuals follow a normal distribution with acceptable deviations. However, the actual vs. predicted values plot reveals significant alignment, showing good predictive accuracy. The externally studentized residuals plot verifies no bias or systematic mistakes. Meanwhile, the residuals vs. run number plot illustrates stable, random swings, demonstrating the model's robustness and independence from data sequence. The SHAP value investigation displayed in Figure 11 demonstrates that "Thickness" exerts the most significant influence on model predictions, as indicated by its elevated average SHAP value. This means that modifications in thickness result in considerable changes in outcomes. On the other hand, X spacing and Y spacing exhibit lesser impacts, with Y spacing suggesting a more stable effect.

Table 12. Performance of different regression models for tridem axle.

Model	Training MSE	Testing MSE	Training R ²	Testing R ²	Training MAE	Testing MAE	Training RMSE	Testing RMSE
Linear Regression	0.020215	0.018016	0.981563	0.990576	0.119391	0.106063	0.142179	0.134224
Ridge Regression	0.020441	0.019307	0.981358	0.989900	0.118321	0.106777	0.142971	0.138950
Lasso Regression	1.096463	1.913948	0.000000	−0.00120	0.846201	1.199572	1.047121	1.383455
Decision Tree	0.000000	0.014616	1.000000	0.992354	0.000000	0.091712	0.000000	0.120899
Random Forest	0.001837	0.013046	0.998324	0.993176	0.029893	0.092359	0.042866	0.114217
Gradient Boosting	0.000068	0.003260	0.999938	0.998295	0.006247	0.037326	0.008222	0.057093
LightGBM	0.060262	0.114631	0.945040	0.940036	0.184756	0.252795	0.245482	0.338571
CatBoost	0.000014	0.115462	0.999988	0.939601	0.002961	0.180794	0.003689	0.339796

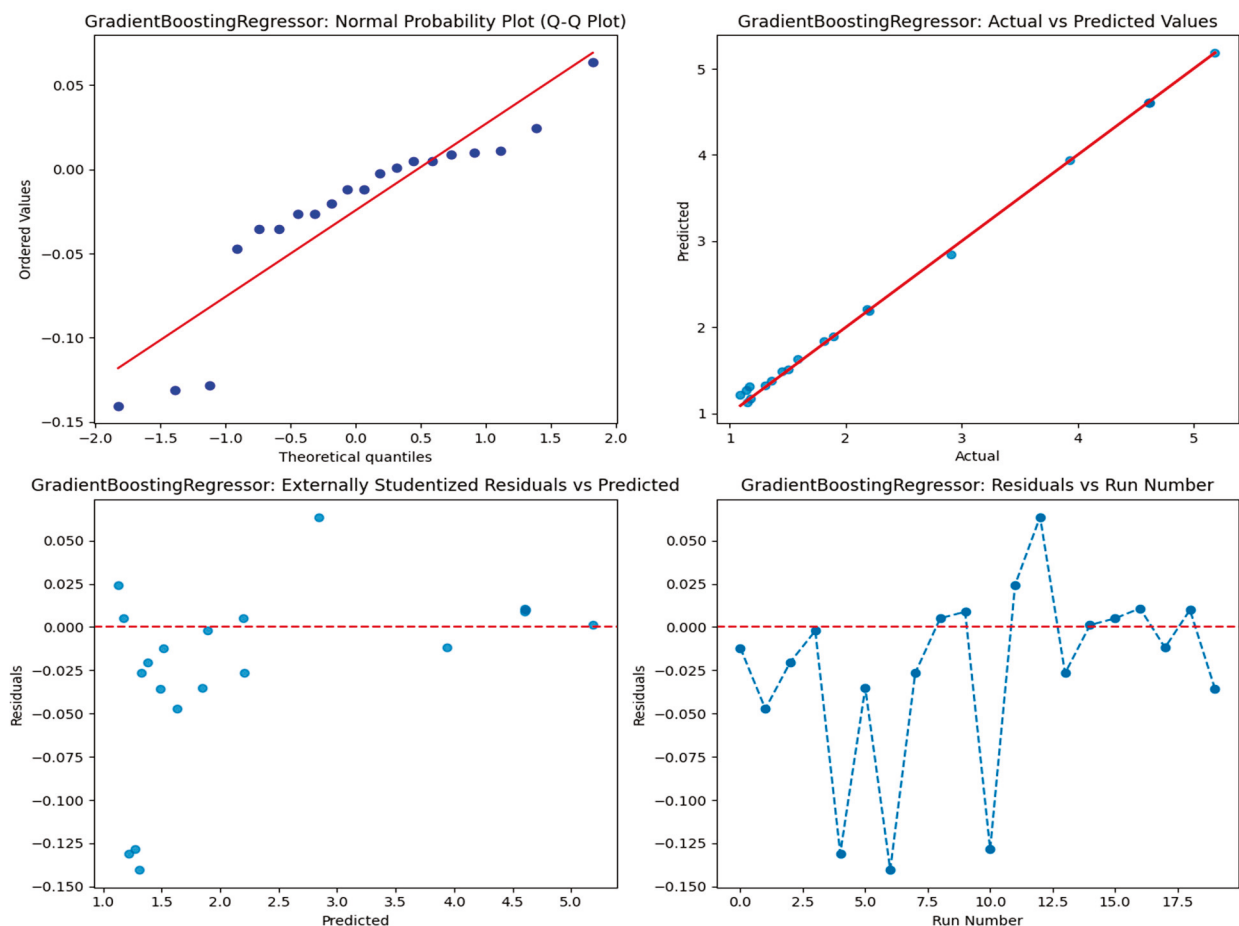


Figure 10. Residuals plots analyzing gradient boosting regression model for tridem axle.

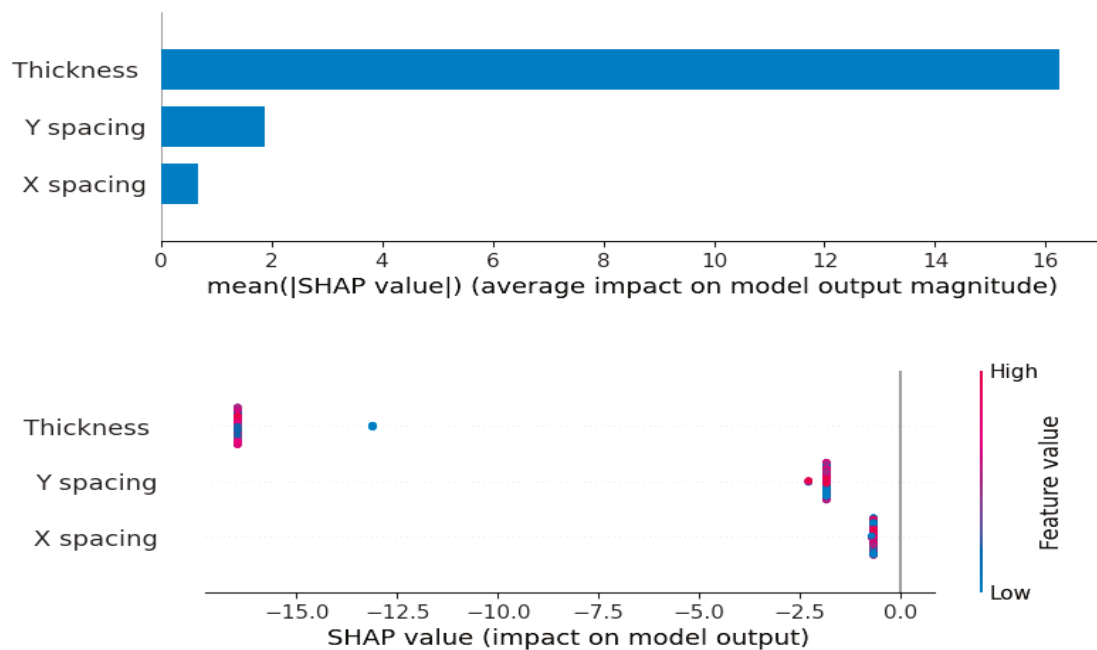


Figure 11. SHAP value analysis for tridem axle model features.

3.3. Discussion

This study analyzes the effectiveness of response surface methodology (RSM) and machine learning techniques in predicting stress levels under various axle configurations, including single, tandem, and tridem axles with dual tires. RSM is proven to be a reliable statistical approach for figuring out correlations between tire spacing, pavement thickness, and stress distribution. The investigation demonstrated that stress levels decrease dramatically with deeper evaluation points, highlighting the significance of depth in pavement design. Additionally, RSM emphasized how reduced tire spacing leads to higher stress levels due to stress overlap, establishing a significant relationship between these variables. Among the RSM models, the quadratic model stood out for its predictive strength, supported by high adjusted R^2 values and statistically significant p -values.

On the other hand, machine learning models, especially gradient boosting, displayed superior prediction accuracy, obtaining the lowest mean squared error (MSE) and root mean squared error (RMSE) across all axle types. However, SHAP value analysis provides a deeper insight on variable relevance. While RSM emphasized the considerable importance of tire spacing on stress levels, machine learning models suggested a lesser function for spacing, with pavement thickness appearing as the most influential element in stress forecasts. This finding highlights how machine learning excels at uncovering complex relationships between variables and pinpointing other factors that influence stress distribution.

While some machine learning models, like decision tree, showed a tendency to overfit, gradient boosting and random forest consistently produced reliable and accurate results. SHAP analysis also provided valuable insights into the importance of various features, showing that, while tire spacing does have an impact, it is not the main factor driving stress effects. Residuals plots offered further validation for the machine learning models by showing that they satisfied crucial assumptions, such as residual independence and consistency.

In conclusion, RSM gives a structured and methodical approach to predicting stress, while machine learning provides the flexibility and precision needed to capture the dynamic nature of pavement behavior. Through implementing these methodologies, researchers can acquire a deeper understanding of pavement performance. RSM can develop initial

hypotheses, and machine learning can enhance these predictions with real-world data. Therefore, this strategy has the potential to customize pavement design, leading to stronger, more durable solutions for a variety of situations while encouraging innovation in engineering techniques.

4. Conclusions

This study aimed to investigate fixed traffic loads, focusing on stress prediction for different axle configurations, including single, tandem, and tridem axles; the conclusions are as follows:

- There is an inverse relationship between the generated stress and the depth of the pavement for single, double, and triple axles. The greater the depth, the lower the stress.
- The spacing between tires in the X and Y directions has a significant effect on the stress. As the spacing decreases, the stress notably increases due to the overlap.
- The use of response surface methodology (RSM) successfully developed models for the generated stress from single, tridem, and triple axles. However, applying a base-10 logarithmic transformation significantly improved the accuracy of the models.
- The quadratic model showed the best performance in predicting stress across all axle configurations. Moreover, it fulfilled all essential statistical assumptions, including normality, homoscedasticity, independence, and linearity, confirming the validity and reliability of the results.
- The analysis of machine learning models showed that gradient boosting showed better performance than the other models in predicting stress.
- Combining RSM and machine learning offers a better understanding of pavement performance. This integrated approach can lead to more effective design strategies, enhancing durability and overall performance.

Future work should focus on multi-layer validation, dynamic loading, and real-world applicability to transition from theoretical models to practical engineering solutions. In particular, future research should investigate both horizontal and vertical stress responses under traffic loads. Collaborations with transportation agencies to acquire field data would add significant value.

Author Contributions: Conceptualization, A.M.A. (Adham Mohammed Alnadish); Methodology, A.M.A. (Adham Mohammed Alnadish) and M.B.R.; Software, A.M.A. (Adham Mohammed Alnadish), M.B.R. and A.M.A. (Aawag Mohsen Alawag); Validation, A.M.A. (Adham Mohammed Alnadish), M.B.R., A.O.B. and A.M.A. (Aawag Mohsen Alawag); Formal analysis, A.M.A. (Adham Mohammed Alnadish) and A.O.B.; Investigation, A.M.A. (Adham Mohammed Alnadish) and A.M.A. (Aawag Mohsen Alawag); Writing—original draft, A.M.A. (Adham Mohammed Alnadish); Writing—review & editing, A.M.A. (Adham Mohammed Alnadish), M.B.R., A.O.B. and A.M.A. (Aawag Mohsen Alawag); Supervision, M.B.R.; Project administration, A.O.B.; Funding acquisition, M.B.R. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: All data generated or analyzed during this study are included in this published article.

Acknowledgments: The authors would like to thank A'Sharqiyah University, Oman, for its support and encouragement of this research. They also wish to extend their heartfelt thanks to the Department of Civil Engineering at NUST Balochistan Campus for its invaluable support.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Bhandari, S.; Luo, X.; Wang, F. Understanding the effects of structural factors and traffic loading on flexible pavement performance. *Int. J. Transp. Sci. Technol.* **2023**, *12*, 258–272. [CrossRef]
2. Adlinge, S.S.; Gupta, A.K. Pavement deterioration and its causes. *Int. J. Innov. Res. Dev.* **2013**, *2*, 437–450.
3. Zeiada, W.A.; Underwood, B.S.; Kaloush, K.E. Impact of asphalt concrete fatigue endurance limit definition on pavement performance prediction. *Int. J. Pavement Eng.* **2017**, *18*, 945–956. [CrossRef]
4. Canestrari, F.; Ingrassia, L.P. A review of top-down cracking in asphalt pavements: Causes, models, experimental tools and future challenges. *J. Traffic Transp. Eng. (Engl. Ed.)* **2020**, *7*, 541–572. [CrossRef]
5. Alencar, G.; de Jesus, A.M.; Calçada, R.A.; da Silva, J.G.S. Fatigue life evaluation of a composite steel-concrete roadway bridge through the hot-spot stress method considering progressive pavement deterioration. *Eng. Struct.* **2018**, *166*, 46–61. [CrossRef]
6. Di Mascio, P.; Moretti, L.; Capannolo, A. Concrete block pavements in urban and local roads: Analysis of stress-strain condition and proposal for a catalogue. *J. Traffic Transp. Eng.* **2019**, *6*, 557–566. [CrossRef]
7. Zeng, Z.; Underwood, B.S.; Kim, Y.R. A state-of-the-art review of asphalt mixture fracture models to address pavement reflective cracking. *Constr. Build. Mater.* **2024**, *443*, 137674. [CrossRef]
8. Al-Janabi, A.J.; Obaid, H.A. Analysis of the Impact of Overloading for Trucks on the Design Life of Flexible Pavement. In *IOP Conference Series: Earth and Environmental Science*; IOP Publishing: Bristol, UK, 2024; Volume 1374, p. 012089.
9. Papagiannakis, A.T.; Masad, E.A. *Pavement Design and Materials*; John Wiley & Sons: Hoboken, NJ, USA, 2024.
10. Huang, Y.H. *Pavement Analysis and Design*; Pearson Prentice Hall: Upper Saddle River, NJ, USA, 2004.
11. Sun, J.; Oh, E.; Chai, G.; Ma, Z.; Ong, D.E.; Bell, P. A systematic review of structural design methods and nondestructive tests for airport pavements. *Constr. Build. Mater.* **2024**, *411*, 134543. [CrossRef]
12. Mahajan, G.R.; Radhika, B.; Biligiri, K.P. A critical review of vehicle-pavement interaction mechanism in evaluating flexible pavement performance characteristics. *Road Mater. Pavement Des.* **2022**, *23*, 735–769. [CrossRef]
13. Clary, L. An examination of the affect of axle spacing on the deformation of thin membrane pavements. In *Masters Abstracts International*; Library and Archives Canada = Bibliothèque et Archives Canada: Ottawa, ON, Canada, 2010; Volume 48.
14. Singh, A.K.; Sahoo, J.P. Analysis and design of two layered flexible pavement systems: A new mechanistic approach. *Comput. Geotech.* **2020**, *117*, 103238. [CrossRef]
15. Huang, Y.H. Computation of equivalent single-wheel loads using layered theory. In *Highway Research Record 291*; Highway Research Board: Washington, DC, USA, 1969; pp. 144–155.
16. Zakaria, N.M.; Yusoff, N.I.M.; Hardwiyono, S.; Mohd Nayan, K.A.; El-Shafie, A. Measurements of the stiffness and thickness of the pavement asphalt layer using the enhanced resonance search method. *Sci. World J.* **2014**, *2014*, 594797. [CrossRef] [PubMed]
17. Ojha, K.N. Flexible Pavement Thickness (A Comparative Study Between Standard and Overloading Condition). *Am. Sci. Res. J. Eng. Technol. Sci. (ASRJETS)* **2019**, *58*, 159–181.
18. Harrell, F.E., Jr.; Harrell, F.E. General aspects of fitting regression models. In *Regression Modeling Strategies: With Applications to Linear Models, Logistic and Ordinal Regression, and Survival Analysis*; Springer: Cham, Switzerland, 2015; pp. 13–44.
19. Alnadish, A.; Aman, Y. Mechanistic approach for reducing the thickness of asphalt layer incorporating steel slag aggregate. *Civ. Eng. J.* **2018**, *4*, 334–345. [CrossRef]
20. Wilches, F.J.; Caballero Guerrero, Á.R.; Patrón Lambraño, G. Modeling of Asphalt Pavement Considering the Application of Empirical and Mechanistic Design Methodologies. *Int. J. Eng. Res. Technol.* **2020**, *13*, 3919. [CrossRef]
21. Fuentes, L.; Taborda, K.; Hu, X.; Horak, E.; Bai, T.; Walubita, L.F. A probabilistic approach to detect structural problems in flexible pavement sections at network level assessment. *Int. J. Pavement Eng.* **2022**, *23*, 1867–1880. [CrossRef]
22. Bueno, B.A.V.; Arenas, M.J.P.; Lugo, A.E.A.; Jaimes, V.E.M.; Ariza, C.A.F.; Chio, G. Deep neural networks for sensitivity assessment of design variables on the structural design parameters of flexible pavements for low-traffic volume roads. *Rev. Colomb. Tecnol. Av. (RCTA)* **2023**, *2*, 122–130.
23. López, R.; Fernández, C.; Pereira, F.J.; Díez, A.; Cara, J.; Martínez, O.; Sánchez, M.E. A comparison between several response surface methodology designs and a neural network model to optimise the oxidation conditions of a lignocellulosic blend. *Biomolecules* **2020**, *10*, 787. [CrossRef]
24. Omranian, S.R. Application of response surface method for analyzing pavement performance. In *Response Surface Methodology in Engineering Science*; Kayaroganam, P., Ed.; IntechOpen: London, UK, 2021; pp. 1–13.
25. Montgomery, D.C. *Design and Analysis of Experiments*; John Wiley & Sons: Hoboken, NJ, USA, 2017.
26. Veza, I.; Spraggon, M.; Fattah, I.R.; Idris, M. Response surface methodology (RSM) for optimizing engine performance and emissions fueled with biofuel: Review of RSM for sustainability energy transition. *Results Eng.* **2023**, *18*, 101213. [CrossRef]
27. Justo-Silva, R.; Ferreira, A.; Flintsch, G. Review on machine learning techniques for developing pavement performance prediction models. *Sustainability* **2021**, *13*, 5248. [CrossRef]
28. Tamagusko, T.; Ferreira, A. Machine Learning for Prediction of the International Roughness Index on Flexible Pavements: A Review, Challenges, and Future Directions. *Infrastructures* **2023**, *8*, 170. [CrossRef]

29. Alnadish, A.M.; Aman, M.Y.; Katman, H.Y.B.; Ibrahim, M.R. Characteristics of warm mix asphalt incorporating coarse steel slag aggregates. *Appl. Sci.* **2021**, *11*, 3708. [CrossRef]
30. Rosopa, P.J.; Schaffer, M.M.; Schroeder, A.N. Managing heteroscedasticity in general linear models. *Psychol. Methods* **2013**, *18*, 335. [CrossRef]
31. Schmidt, A.F.; Finan, C. Linear regression and the normality assumption. *J. Clin. Epidemiol.* **2018**, *98*, 146–151. [CrossRef]
32. James, G.; Witten, D.; Hastie, T.; Tibshirani, R.; Taylor, J. Linear regression. In *An Introduction to Statistical Learning: With Applications in Python*; Springer International Publishing: Cham, Switzerland, 2023; pp. 69–134.

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

Review

Advancing Efficiency and Sustainability in Road Construction: A Bibliometric Review of Recent Innovations and Challenges

Kornel Nagy ¹, Bernadett Bringye ^{1,*} and Zoltan Karoly Lakner ^{2,3}

¹ Institute for Rural Development and Sustainable Economy, Hungarian University of Agriculture and Life Sciences, 2100 Gödöllő, Hungary; nagy.kornel@uni-mate.hu

² Institute of Agricultural and Food Economics, Hungarian University of Agriculture and Life Sciences, 2100 Gödöllő, Hungary; lakner.zoltan.karoly@uni-mate.hu

³ Department of International Relationships, Tashkent State Agrarian University, Tashkent 100140, Uzbekistan

* Correspondence: bringye.bernadett@uni-mate.hu; Tel.: +36-28-522-000

Abstract

It is well documented that road construction is a pillar of well-balanced socioeconomic development worldwide. The first decades of the new millennium have witnessed unprecedented development in road construction activities in emerging economies and the Global South. At the same time, the construction industry is widely considered to be a rather conservative one, based on traditional technologies and materials. However, the development of materials science increases the possibilities and volumes of by-products from various technologies, and the selective collection of garbage necessitates innovation in the road construction sphere. The goal of this paper is to provide a broad overview of innovations in road construction. Based on a bibliometric approach, the article analyses the various trends in road construction, where the increasing pressure to reduce costs and the environmental footprint drives deep-rooted innovation through the use of new materials and the optimisation of technologies and management methods. Our results highlight the potential for significant improvements in road construction efficiency, environmental impact, and cost-effectiveness through the adoption of these technologies and methodologies, as well as a trend towards more efficient, sustainable, and technologically advanced road construction practices, with a focus on overcoming traditional inefficiencies and environmental concerns. Future research should continue to focus on addressing these challenges and developing comprehensive, adaptable solutions for the road construction industry, while leveraging the latest findings in this area.

Keywords: road construction; efficiency; sustainability; environmental impact

1. Introduction

Road construction is a critical area of the construction sector due to its fundamental role in establishing infrastructure, supporting economic growth, and shaping society and the environment. Roads are essential for economic development as they facilitate logistics: they provide access to markets and connections between facilities. Improved road infrastructure can boost the GDP and economic performance [1]. The “Belt and Road Initiative” has shown that infrastructure construction can promote economic growth and improve income distribution along the route, particularly in developing countries [2]. Multiple factors contribute to economic growth: expenditure on road construction supports the private sector with funds, promotes production, and stimulates the economy. This increases profits and, through them, taxes, which boost the government. Access through

roads increases property values and property taxes, enabling local authorities to invest more in infrastructure [3]. Similarly, a lack of new developments or delays, or overruns on such projects, might lead to losses, missed opportunities, and the degradation of society [4].

Construction is one of the least digitised sectors, and road construction is particularly lagging [5]. Meanwhile, other sectors have worked systematically to improve their efficiency, standardising and automating processes where appropriate, which has led to significant increases in productivity over the past few decades. A study was published on the implementation of Industry 4.0 technologies and lean-based flow optimisation principles [6]. Advanced project management and modelling techniques [7] can be enhanced by the use of building information modelling (BIM), which enhances overall efficiency [8]. BIM modules can be leveraged to accelerate approval processes [9]. Determining factors of inaccurate estimates—local wishes, defective estimations, long processing times during planning, scope changes, market condition changes, and unforeseen conditions—helps to reduce cost overruns [10]. Thoroughly detailed scheduling at the activity level helps to develop more realistic schedules and prevent overruns [11]. The transportation of road construction materials significantly contributes to costs. Planning routes and reducing stops, accelerations, and idle times reduces costs, and artificial neural networks could be leveraged for optimisation [12]—further optimising machinery and coordination, simultaneously operating machines through hydraulic pumps. Reducing the number of pumps improves efficiency [13].

Technological innovation can improve quality and durability and help to lower the carbon footprint. Research into new materials such as self-healing concrete, prefabricated and recycled plastic roads, and solar roads can lead to more durable and sustainable roadways [14]. These innovations can reduce maintenance costs, extend the road's lifespan, and thus reduce overall lifecycle costs. Before new technologies are widely implemented, they need to be rigorously tested to minimise risks. Programs like Germany's duraBASt provide a platform for such testing [15]. Road infrastructure and urbanisation are interconnected, with road network expansions significantly influencing socioeconomic development from urban to rural areas [16].

Road construction has significant environmental impacts, including CO₂ emissions and resource consumption [17]. Research into sustainable construction practices, such as recycling materials and using energy-efficient technologies, is crucial in mitigating these impacts [18]. Comparative lifecycle analyses of construction methods help to understand environmental impacts and promote eco-friendly practices [19].

Planning considerations in urban areas are also popular areas of research; reducing congestion through certain simulations might be an important consideration [20]. As cities expand their borders, forming urban agglomerations with neighbouring settlements, road development establishes connections between them [21]. Improvements in infrastructure increase population inflows [22]. Smart roads in the "Smart Age" leverage the latest innovations in this area [23].

The motivation for this bibliometric analysis is to provide an overview of the broader perspective, identify similar papers, and help to combine them to respond to ESG expectations and mitigate challenges through improving efficiency.

2. Materials and Methods

Our analysis aims to consolidate the current state of knowledge in road construction, based on a bibliometric analysis of scientific articles published until mid-2025. Similar reviews on this topic have been published (Table 1) from a slightly different perspective [16,24,25]. This article's main novelty lies in using a targeted bibliometric approach to map how efficiency and sustainability are jointly addressed in recent road construc-

tion research and then linking these patterns to concrete technological and managerial directions for future work. Beyond descriptive bibliometrics, the study interprets thematic clusters as concrete “innovation avenues” for improving both efficiency and environmental performance in road projects. This connection between quantitative mapping and practice-oriented recommendations (implementation, standardisation, and policy implications) provides an original synthesis that goes beyond earlier reviews focused solely on materials, sustainability, or project management in isolation.

Table 1. List of similar review articles.

Authors	Title	Scope	Method	Limitations	How This Fills the Gap
(Alnadish et al., 2024) [24]	A Bibliometric Analysis and Review on Applications of Industrial By-Products in Asphalt Mixtures for Sustainable Road Construction.	A bibliometric analysis of industrial by-product usage in asphalt mixtures	Bibliometric	Asphalt mixtures only, until December 2022. VOSviewer used (They used version 1.6.18).	Our review has a wider scope, including asphalts and other innovative materials, digitisation, and management techniques, and uses more advanced tools.
(Mhana et al., 2024) [16]	Road urban planning sustainability based on remote sensing and satellite dataset: A review.	Review on remote sensing and satellite-based methods used to analyse road networks, land use/land cover change, urbanisation, and their environmental and road safety impacts.	Narrative	VOSviewer used for keyword analysis. Deeper emphasis on certain methods rather than bibliometric data.	Our review has a wider scope; it focuses more on bibliographic analysis while using more advanced tools.
(Pentury et al., 2025) [25]	Application of digitalisation and computerisation technology in road construction.	Review on digitalisation and computerisation technology applications in road construction.	Systematic/bibliometric	Focuses on digital/computerisation applications in road construction.	Our review has a wider scope, including digitisation but also innovative materials and management aspects. Similar tools used.

This review focuses on the following questions.

Q1: What is the current state of academic inquiry into road construction efficiency, automation, and digitalisation? Which publications, researchers, institutions, and countries are most active in this area, and how are collaborative relationships and research topics interconnected within the scholarly community?

Q2: Which major topics and recurring issues define scientific studies on efficiency, automation, and digital transformation in road construction? How can these topics be systematically grouped into foundational drivers, essential and interdisciplinary concepts, emerging research directions, and niche or marginal areas within the field?

Q3: Given recent advancements and ongoing developments, which future directions and research opportunities are likely to shape the evolution of road construction efficiency, automation, and digitalisation in the coming years?

We used the Preferred Reporting Items for Systematic Reviews (PRISMA) method. Limitations: Web of Science and Scopus were used only as sources. Only English publications were considered. An uneven number of reviewers voted when deciding about

articles to include, namely a team of 3. The team consisted of two economists and an engineer–economist; the relative heterogeneity offered an excellent opportunity to compare and analyse the various approaches.

The initial search used a broad query combining multiple dimensions of efficiency, sustainability, road-related infrastructure, materials and resource use, and various impact terms, which returned 2097 records and included many marginally relevant papers. The search queries included the following: ALL = (((“efficiency” OR “effectiveness” OR “productivity” OR “performance”) AND (“sustainability” OR “sustainable” OR “eco-friendly” OR “green”) AND (“road construction” OR “road building” OR “pavement” OR “infrastructure”) AND (“materials” OR “resources” OR “techniques” OR “methods”) AND (“waste reduction” OR “energy efficiency” OR “cost savings” OR “environmental impact”))). While “pavement” and “infrastructure” gave broader results than expected, the second and last parts of the query restricted it too much.

To sharpen the focus, the search strategy was narrowed by restricting the time window to the last ten years and simplifying the query to target only publications explicitly linking “efficiency” with “road construction”, “high-speed road networks”, or “motorway construction”, thereby excluding studies that mentioned the earlier sustainability or materials terms only tangentially. This deliberate narrowing allowed the review to isolate how productivity, costs, and energy efficiency are treated together with sustainability, rather than including general pavement or environmental studies.

This refinement reduced noise, removed irrelevant or weakly related results, and yielded a more manageable and thematically coherent set of 519 documents (Figure 1), which were then used for subsequent bibliometric analysis and visualisation in R (Bibliometrix version 4.0) and VOSviewer (version 1.6.20).

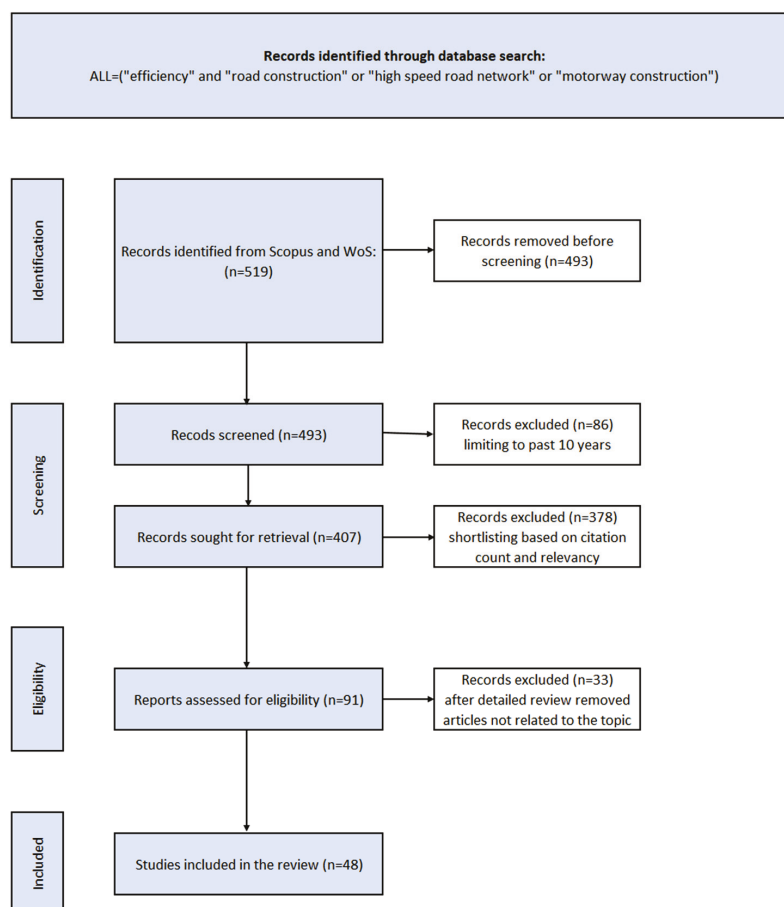


Figure 1. Process of paper selection.

3. Results

The earliest document found was published in 1975. Figure 2 shows the annual production of the articles. The jump in 2016, the dip in 2022, and the very high value in 2025 are best interpreted as artefacts of the bibliometric data and global shocks (mainly COVID-19), rather than sudden structural changes in road construction science itself. Many road construction subfields (work zones, waste reuse, sustainability, emissions) show general acceleration in publications around the mid-2010s, driven by cheap sensing, BIM/ITS data, and sustainability policy agendas. Many 2014–2016 articles in conferences and journals only became fully indexed in 2016, so a step increase in coverage is expected when new series or categories (e.g., “Green and Sustainable Technology”, “Environmental Sciences”) are added to the search query or to databases. Bibliometric reviews of transport and construction topics report small but visible drops in 2020–2021 outputs, attributed to COVID-19 disruptions to labs, field work, and infrastructure projects. In our results, the decrease is delayed to 2022, which fits the pattern, because many 2020–2021 manuscripts were already in the pipeline and appeared on schedule, while project delays, funding cuts, and postponed conferences affected the 2022 publication cohort the most severely (especially proceedings). The rapid growth in 2024 and the spike in 2025 reflect a return from pandemic slowdowns plus accumulated delayed work and the rapid expansion of niches such as recycling in pavements, low-carbon binders, and lifecycle assessment for roads, all of which are heavily represented in the attached records. As 2025 is the last fully included year in your dataset, “online first” and early-access items are grouped into this year in WoS/Scopus exports.

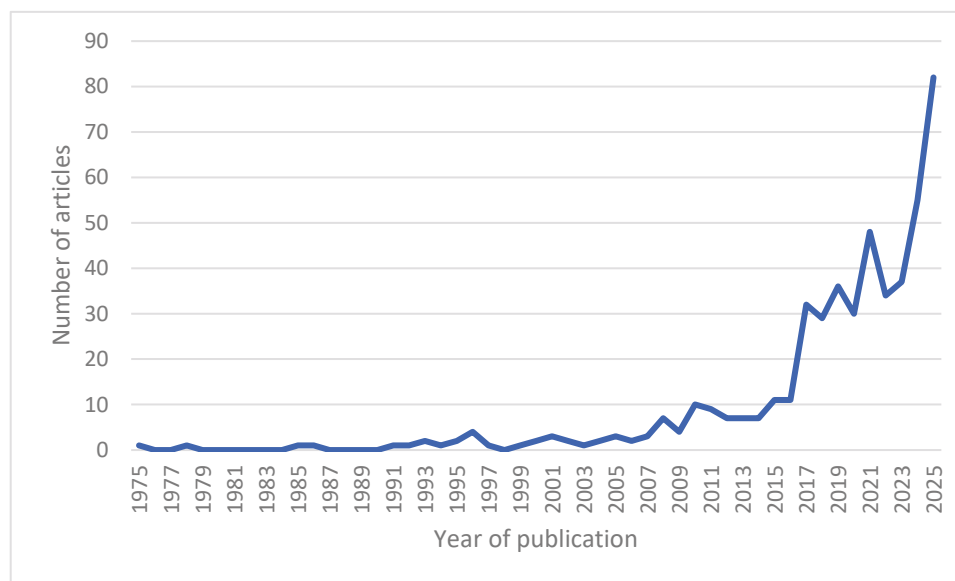


Figure 2. Quantity of annual production on road construction until end of 2025—created using R Bibliometrix (n = 493).

The Sankey diagram (Figure 3) links individual authors to their countries and then to their primary keywords, showing how national communities contribute to different road construction topics. On the left, each bar represents a prolific author in the dataset (e.g., Odeck J., Jongpradist P., Liu Q., Wang S.Y., Han T., Van den Bergh W.). The width of each flow to the middle column indicates how strongly the author’s output is associated with a country, with China clearly dominating the central column, followed by Iran, Australia, Poland, the USA, Russia and several European countries. From the country column, flows run to the right-hand column of author keywords, including road construction,

sustainability, asphalt pavements, efficiency, stabilisation, the California bearing ratio, reclaimed asphalt pavements, the circular economy, machine learning, and hot-mix asphalt. Thick links from China and Iran are seen for topics such as road construction, sustainability, asphalt/asphalt pavements, and efficiency.

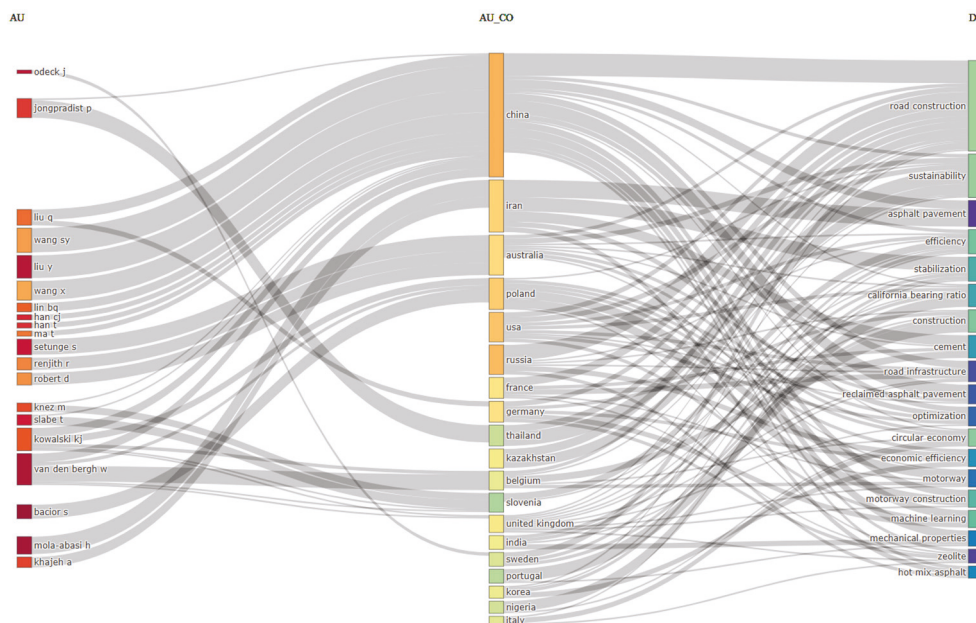


Figure 3. Three-field plot: authors, keywords, and countries—created using R Bibliometrix (n = 493).

In contrast, European countries (e.g., Belgium, Slovenia, Germany) connect more strongly to topics such as motorway construction, efficiency analysis, and the mechanical properties or stabilisation of materials. The diagram highlights that Chinese-affiliated authors anchor much of the literature on general road construction and sustainability. At the same time, a diverse set of countries contributes to specialised topics such as the circular economy, machine learning-based pavement design, and economic efficiency.

In total, 2707 keywords were found, and 80 had at least five occurrences, as shown in Figure 4. Road construction, efficiency, and related terms had been removed from the network view because they were part of the search keywords and were expected to occur frequently. Removing them helped to illustrate the other keywords better. Their numbers were as follows: performance (31), concrete (21), model (19), impact (18). They form five clusters and have 547 links, and the total link strength is 758. The red cluster contains terms such as productivity, impact, economic efficiency, growth, optimisation, simulation, data envelopment analysis, BIM, GIS, monitoring, motorway construction, and water quality. This cluster captures management- and policy-oriented work on transport infrastructure: performance measurement, investment and productivity analysis, the optimisation of road networks, and broader sustainability/monitoring issues. The green cluster includes lifecycle assessment, the circular economy, emissions, energy, environmental impacts, greenhouse gas emissions, impacts, removal, temperature, hot-mix asphalt, bitumen, reclaimed asphalt, mixtures, and water. It represents research on the environmental performance of pavements and materials, especially LCA-based evaluations of asphalt technologies, emissions, and waste reuse within a circular economy framework. The blue cluster contains aggregate, cement, lime, soil, fly ash, stabilisation, durability, strength, zeolite, waste, mechanical properties, and related terms. This cluster reflects experimental and modelling work on the mechanical behaviour and stabilisation of road construction materials, particularly binders, soil improvement, and the durability/strength of pavement layers. The yellow cluster includes algorithm, prediction, machine learning,

design, cost, the California bearing ratio, compressive strength, resilient modulus, slag, asphalt pavement, asphalt mixtures, and RAP (reclaimed asphalt pavement). It groups methodologically focused studies that use algorithms and machine learning or optimisation techniques to predict and design the performance of asphalt layers and recycled mixtures, with strong links to the blue and green material clusters. The purple cluster is focused on asphalt, concrete, and performance and is closely linked to lifecycle assessment and environmental impacts. This small but central cluster acts as a bridge between the environmental (green), mechanical (blue/yellow), and management (red) groups, representing mainstream pavement performance studies that connect material choices with broader sustainability and efficiency outcomes.

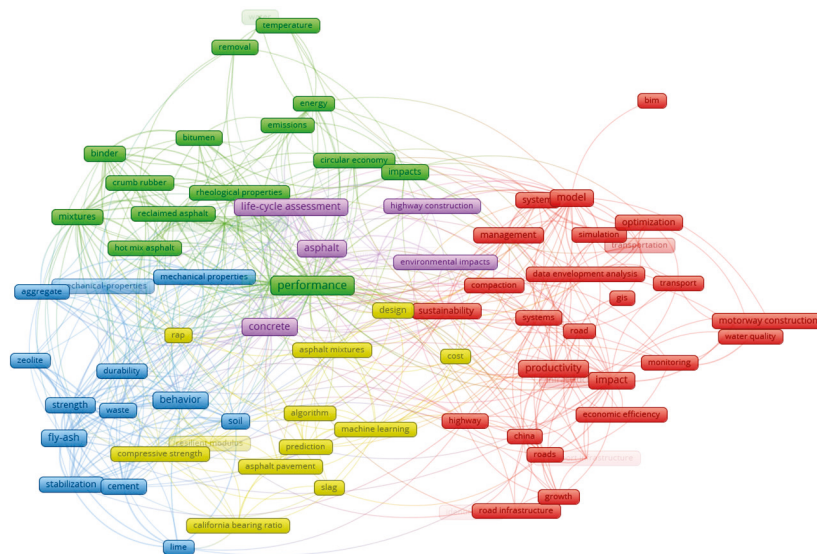


Figure 4. Keyword co-occurrence—created using VOSviewer (n = 493).

The most relevant sources (Figure 5) show that a small group of journals accounts for most of the publications on the road construction topic, with the clear dominance of material- and sustainability-oriented outlets. *Construction and Building Materials* is by far the leading source, with the largest bubble and highest document count (18 papers), confirming it as the primary venue for road material and pavement research in the dataset. *Applied Sciences—Basel* follows at some distance (about 14 documents), acting as a broad engineering journal that still contributes substantially to this field. *Sustainability*, the *Journal of Cleaner Production*, and *Buildings* form a strong second tier, each with roughly 6–10 contributions, indicating that environmental performance, LCA, and sustainable construction are major dissemination channels for road construction work. *Environmental Science and Pollution Research* and *Science of the Total Environment* appear with smaller bubbles, showing that a non-trivial portion of road construction studies is published in environmental science outlets rather than pure civil engineering journals. *Case Studies in Construction Materials*, the *Journal of Materials in Civil Engineering*, and *Engineering Geology* have fewer documents but still register as key specialised venues for detailed material behaviour and geotechnical aspects. The distribution suggests that the topic sits at the intersection of pavement/materials engineering and sustainability/environmental assessment, with *Construction and Building Materials* as the central hub.

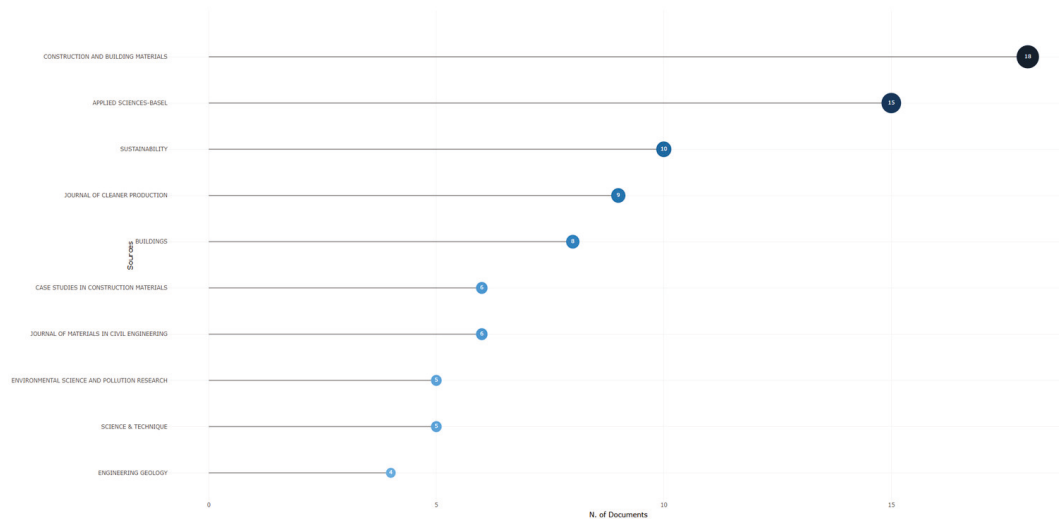


Figure 5. Most relevant sources—created using R Bibliometrix (n = 493).

The author charts (Figures 6 and 7) show that a handful of core authors contribute modest but sustained outputs, while several newer authors have short, recent bursts of publications. The bar/bubble chart (Figure 6) indicates that four authors (Bacior S., Mola-Abasi H., Van den Bergh W., Liu Y.) are the most prolific, each with five documents, followed by Odeck J., Setunge S., and Slabe T. with four, and three newer authors (Han C.J., Han T., Jongpradist P.) with three each. Productivity per author is relatively low (a maximum of five papers), suggesting a dispersed authorship pattern in which no single researcher dominates the field. The timeline chart (Figure 7) shows that Odeck J. and Slabe T. have the most extended publishing spans, contributing sporadically from the mid-1990s or early 2000s to around 2020, while Setunge S.’s work also spans more than a decade. In contrast, the most prolific recent authors (Bacior S., Mola-Abasi H., Van den Bergh W., Liu Y., Han C.J., Han T., Jongpradist P.) show publications clustered within the last decade, particularly after 2015, mirroring the overall growth of road construction research.

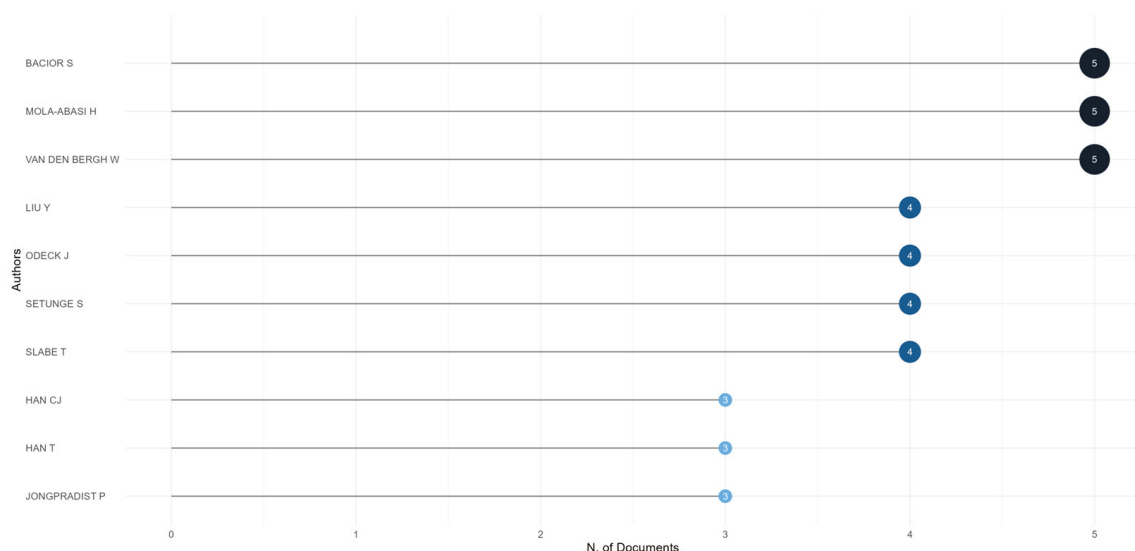


Figure 6. Most relevant authors—created using R Bibliometrix (n = 493).

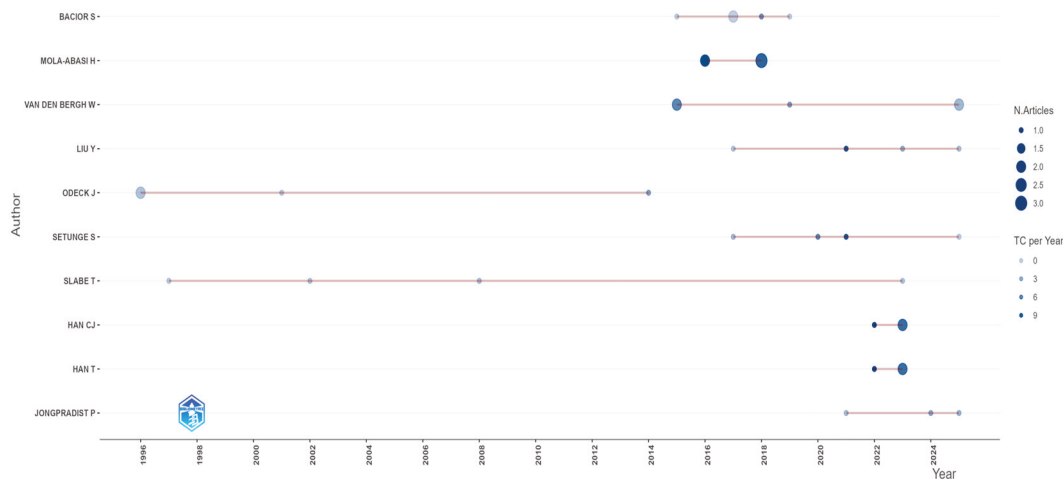


Figure 7. Authors' productivity over time—created using R Bibliometrix (n = 493).

In the corresponding countries ranking (Figure 8.) China has by far the highest number of documents (well over 120), with only a small dark segment at the end of the bar, indicating that most papers are single-country publications (SCP) and only a minority are multi-country collaborations (MCP). The USA, Australia, the United Kingdom, Germany, and several European countries show shorter bars but relatively larger dark segments, meaning that a higher share of their papers involve international co-authorship. Russia is the second-largest producer after China, yet its bar is almost entirely light blue (SCP), with very few MCP, consistent with the reduced international collaboration intensity in recent years. Iran's output is mid-range but only SCP, suggesting that sanctions and political isolation limit joint projects with foreign partners, especially in infrastructure-related fields. Countries such as Poland, Australia, the UK, Kazakhstan, and some Western European states have moderate outputs but visibly higher MCP proportions, indicating that they act as collaboration hubs connecting to the larger producers. Amongst the larger countries, China has a relatively high number of MCP; the USA, Australia, and the UK similarly show a concentration of volume in a few mostly domestic systems, while the figures for Russia and Iran might be related to their geopolitical situations. Many smaller producers are more open to international partnerships to contribute to road construction research. We can see SCP in Norway, Belgium, and Slovenia as well, which are not isolated—this is probably due to the low number of publications (5–6 per country). The overall MCP ratio is relatively low; more international cooperation should be encouraged.

The thematic map generated from the bibliometric analysis divides the research landscape of road construction into four principal quadrants based on development and relevance. To illustrate the pace of development in the topic, we ran the analysis with the same queries on 12 August 2025 and six months later on 12 January 2026 (Figures 9 and 10). The latter thematic map shows that sustainability- and efficiency-related topics have become more central and coherent, while several earlier themes have become fragmented or moved to the periphery. Motor themes earlier were dominated by “impact–productivity–growth”, the lifecycle assessment of asphalt mixtures, and performance/efficiency models. Six months later, these ideas were concentrated into larger, clearer clusters around “concrete–pavement–fly ash”, “CO₂ emissions–highway–economic growth”, and a still-dominant “performance/efficiency model”, indicating a stronger focus on low-carbon materials and macroeconomic/CO₂ issues alongside efficiency. Basic themes initially included “road models–urban form”, “land use–network–accessibility”, “finite element analysis”, and generic “cost reduction”. In the newer map, basic themes shift toward more technical and applied clusters such as “algorithm–prediction–compressive strength”, “road

models–networks”, “removal–water–heavy metals”, and “cost reduction–flow”, suggesting that modelling and treatment techniques have become standard supporting tools for the dominant sustainability agenda. Previously, niche themes like “flow criteria–projects” and “combustion–emission characteristics” were relatively isolated but still linked to machinery and project-level issues. Later, niche themes included “key parameters–retardation factor”, “finite element analysis–implementation–propagation”, and hydrological clusters (“criteria–process–runoff”, “capillary–design–classification”), which are methodologically dense but even less central, indicating deeper specialisation and weaker connection to mainstream road construction debates. In both maps, the “vegetation” topic sits in the emerging/declining quadrant, with some minor shifts but no move toward centrality. This persistence suggests that safety- and ecology-related topics remain secondary compared with the rapidly consolidating low-carbon, waste reuse, and performance/efficiency lines that are driving the global growth in publications.

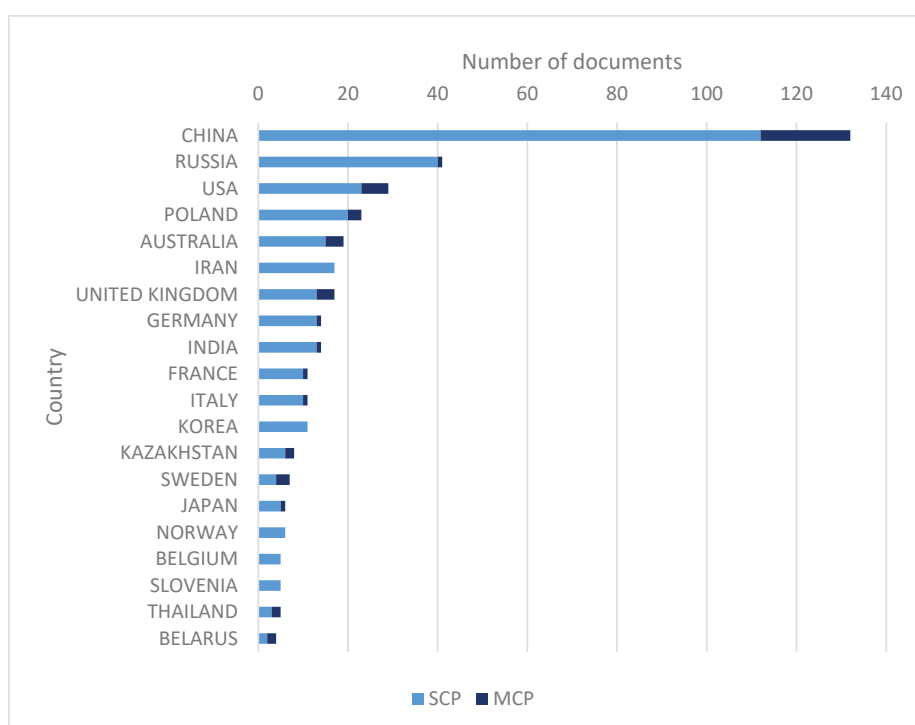


Figure 8. Corresponding authors’ countries, including single-country (SCP) and multi-country (MCP) publications—created using R Bibliometrix (n = 493).

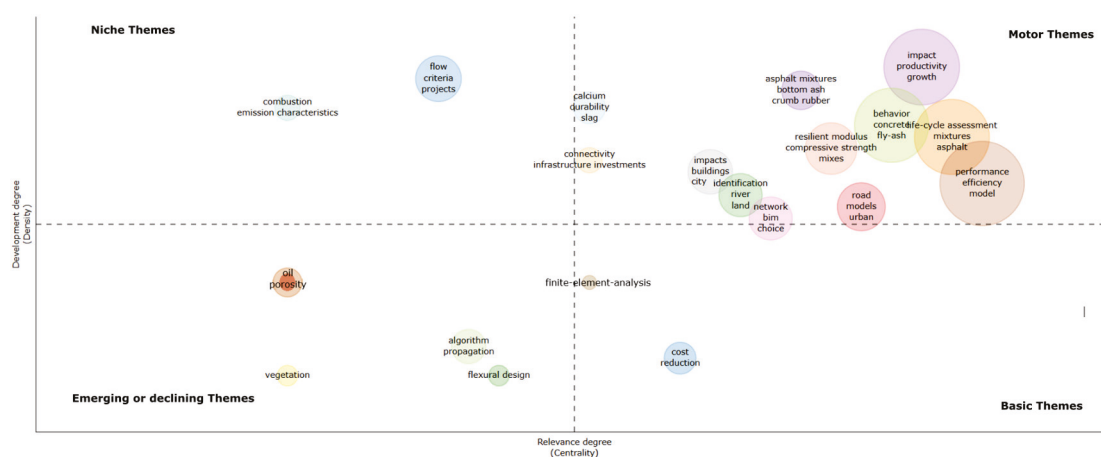


Figure 9. Thematic map created using R Bibliometrix, run on 12 August 2025 (n = 469).

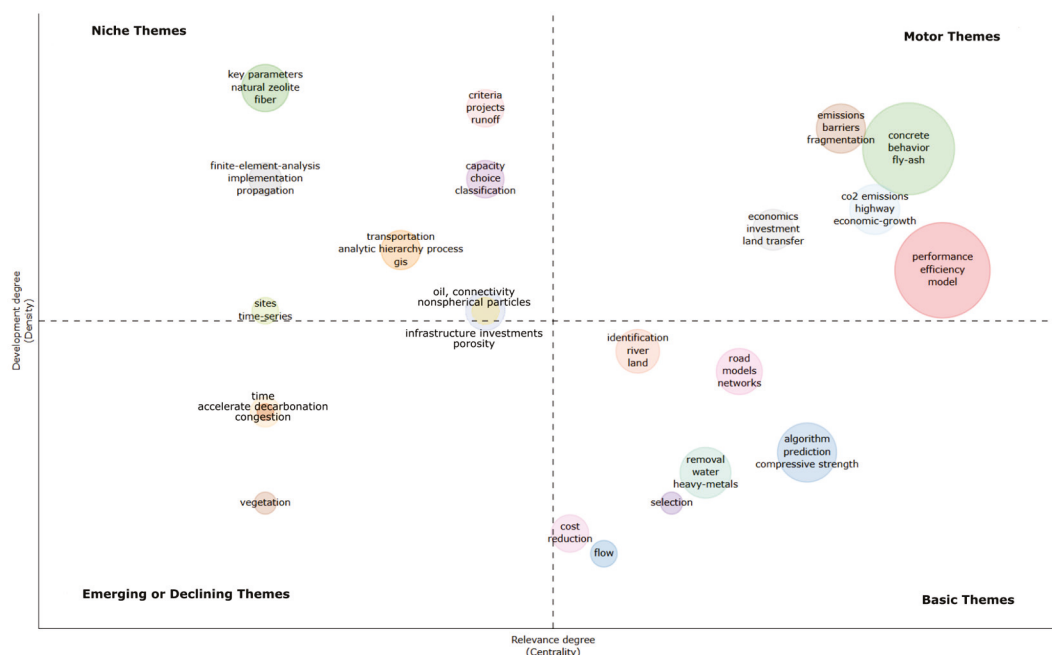


Figure 10. Thematic map created using R Bibliometrix, run on 12 January 2026 (n = 493).

3.1. Findings of Most Relevant Papers

The research in the most relevant papers focused on multiple areas: the use of innovative materials (binders, stabilisers), management (accelerations, efficiencies, optimisations), digitisation (BIM, drones/optical recognition), other innovations (smart cities/smart roads, smart design), and the application of artificial intelligence or digital twins. The most cited ones (Table 2) are mostly from research on innovative/waste materials.

Table 2. List of the relevant journals that have been cited the most frequently.

Journal (JCI)	Authors	Title	Number of Citations	Category
<i>Resources, Conservation and Recycling</i> (1.56)	(Pana et al., 2019) [26]	A review on lead slag generation, characteristics, and utilisation	115	Innovative materials
<i>Sustainable Cities and Society</i> (1.95)	(Xiao et al., 2022) [27]	Exploring the coupling coordination and key factors between urbanisation and land use efficiency in ecologically sensitive areas: A case study of the Loess Plateau, China	110	Management
<i>Construction and Building Materials</i> (1.41)	(Fang et al., 2021) [28]	Comprehensive re-view on the application of bio-rejuvenator in the regeneration of waste asphalt materials	105	Innovative materials
<i>Journal of Cleaner Production</i> (1.54)	(Bamigboye et al., 2021) [29]	Waste materials in highway applications: An overview on generation and utilisation implications on sustainability	103	Innovative materials

Table 2. Cont.

Journal (JCI)	Authors	Title	Number of Citations	Category
<i>Construction and Building Materials</i> (1.41)	(Zentar et al., 2021) [30]	Comparative study of stabilisation/solidification of dredged sediments with ordinary Port-land cement and calcium sulfo-aluminate cement in the framework of valorisation in road construction material	96	Innovative materials
<i>Journal of Cleaner Pro-Duction</i> (1.54)	(Mah et al., 2018) [31]	Life cycle assessment and life cycle costing toward eco-efficiency concrete waste management in Malaysia	84	Innovative materials
<i>Cement & Concrete Composites</i> (2.02)	(Zhao et al., 2018) [32]	Use of uncontaminated marine sediments in mortar and concrete by partial substitution of cement	79	Innovative materials

3.2. Innovative Materials

This is the most interesting topic in terms of the number of citations: eight out of the top 10 cited articles belong here. This includes binders, soil stabilisers, cement alternatives, and the use of waste materials from other industries, such as fly ash or bottom ash. Xanthan gum (XG) was used as a stabiliser for road shoulders in a Sri Lankan construction case [33]. They compared it with conventional cement-, fly ash-, and bottom ash-blended binders as well [34]. After a slightly longer curing time (28 days), XG-treated soil had remarkably higher unconfined compressive strength (UCS) and also higher ductility. Ref. [29] compared the use of waste materials in a review paper. They compared PET (plastic bottles), waste rubber or crumb rubber (CR) derived from end-of-life tires (ELT), steel slag, glass waste, construction and demolition waste (CD), recycled asphalt pavement (RAP), recycled asphalt shingles (RAS), colliery spoils (CS), fly ash (FA) as a by-product of coal combustion, cement kiln dust (CKD), aluminium dross, mine tailings, bio-oils (from agricultural and organic wastes), and incineration bottom ash (incineration of solid wastes). In their results, PET, CR, RAP, RAS, steel slag, and bio-oils were used in asphalt layers; all others were used for ground improvement or in pavement sublayers. CD wastes were very efficient in landfills. The utilisation of waste materials can improve recycling rates, effectively reduce landfill requirements, preserve natural aggregates from depletion, and significantly reduce costs. It was also noted that waste materials must be carefully tested and free of hazardous substances to prevent environmental contamination [29,32].

Lateritic soil, due to its availability and affordability, is one of the most popular building materials in tropical and sub-tropical regions. Binders require most of the energy, thereby mainly contributing to costs and CO₂ emissions. Using low-energy, alternative pozzolan geopolymers for soil/gravel stabilisation improves efficiency by reducing costs and carbon emissions [35]. While producing pozzolans requires high temperatures (1400 °C), their alternative binder needs only 100 °C. Enzymes used for soil stabilisation are also an opportunity [36]. Alternative cement (calcium sulfo-aluminate—CSA) was compared to ordinary Portland cement (OPC) by [30]. CSA is green, low-carbon, and environmentally friendly: its limestone demand is 35–40% lower than that of OPC, and the production temperature is 200 °C lower, resulting in 33% lower CO₂ emissions, while CSA's compressive strength is 78–98% that of OPC.

The performance of three-layer reinforced concrete plate constructions was analysed in [37], focusing on their load-bearing capacity, crack resistance, and deformation under varying reinforcement percentages and concrete grades. The authors found that reducing the structural weight by using a lightweight, self-sealing expanded clay concrete middle layer can significantly improve the energy efficiency and economic benefits. They experimentally tested 16 plate samples to show how factors like the concrete grade and reinforcement affect their strength and durability, aiming to optimise construction materials for transport buildings and road construction.

Composite cationic bituminous structures, a novel material offering several advantages over traditional hot and cold asphalt paving, were studied in [38], with findings such as their usability in various weather conditions, reduced bitumen usage, quicker road reopening, and enhanced durability. The study developed price formation schemes and estimated specifications for road works with this technology, demonstrating significant economic and ecological benefits, including cost reductions and more durable road surfaces compared to conventional methods. The findings highlight the potential of this composite technology to improve the working efficiency and environmental impacts in road repair projects. Incorporating reclaimed asphalt pavement (RAP) and recycled concrete aggregates (RCA) also significantly reduces the environmental impact [39] by conserving natural resources and minimising waste [40], and this was also studied in an earlier paper [41].

Pulp mill fly ash (PFS) dramatically improves soil strength and stiffness due to its self-cementing properties, and its application minimises the bioavailability of toxic metals, making it suitable for sustainable construction and reducing industrial waste. One study's findings suggest that PFA can be effectively implemented as a green binder in road and pavement applications, supporting both ecological and economic benefits [42].

3.3. Management

The system of systems (SoS) concept has been studied for improved road construction in [6]. According to their findings, on-site execution and coordination, such as improved planning, communication, Internet of Things (IoT)-based analytics, and the use of building information modelling (BIM), could raise productivity by 20–25%. Potential waste reduction and improved machine movements can reduce fuel consumption by 31%. SoS could be applied primarily in the planning phase. A detailed activity-based scheduling model was developed in [11]. MATLAB (version 2023b, MathWorks, Natick, MA, USA) algorithms (DADER and DP) were used during the planning of the construction. The duration of the project without applying the model was 3913 days, and, with the model, it was reduced to 2431 days. The total cost remained nearly identical, but the acceleration was significant, down to 60%. Further schedule optimisation is possible through accelerating approvals [9]. In their quantitative study, the Common Data Environment (CDE) as a digital platform was compared to conventional methods. Using the CDE was considered to have high value added, especially in terms of transparency and traceability, which were identified as most concerning in the conventional methods. The implementation of the CDE cross-country and with public participation is a future research topic; the use of BIM could be an opportunity. Promising added value in planning and approval processes is predicted in this context.

A novel hybrid approach combining egalitarian bargaining game theory, data envelopment analysis (DEA), and sustainable network design to evaluate, select, and schedule urban road construction projects has also been studied, with an application to the Isfahan city network. The proposed model integrates sustainability (economic, social, and environmental aspects) and controls undesirable shocks in network performance across multiple periods, using a meta-heuristic genetic algorithm to solve large-scale instances

efficiently. Results from both simulated and real-world cases demonstrate that the method enhances network efficiency, sustainability, and stability while optimising project selection and timing within technical and financial constraints [43].

An evaluation method for municipal road construction quality was developed that incorporates carbon emissions to ensure environmentally friendly construction [44]. This method uses an improved genetic algorithm (GA) combined with a spanning tree constraint approach to optimise the assessment of road construction flow parameters, accounting for factors such as material quality and environmental impact. Experimental results demonstrate that this method is accurate, reliable, and achieves a smaller carbon footprint compared to existing methods, aligning with the priority of environmental protection in infrastructure development.

To reduce planning costs, fuel consumption might be identified as an improvement area, as in [12]. The authors developed a model using artificial neural networks, focusing on optimal driving speeds, routes, and road conditions. Mitigating idle time and reducing the number of stops also help to reduce fuel consumption.

Determining factors of inaccurate estimates—local wishes, defective estimations, long processing times during planning, scope changes, market condition changes, and unforeseen conditions—helps to reduce cost overruns [10]. The transportation of road construction materials significantly contributes to costs. Further optimising machinery could involve coordinating the simultaneous operation of machines via hydraulic pumps. Reducing the number of pumps improves efficiency [13].

3.4. Digitisation

Digitisation helps to manage projects, provide transparency, and support decision-making, as discussed in the previous section. In this section, the focus is on the engineering application. Research interest in BIM, 3D models, and the use of GPS and unmanned aerial vehicles (UAVs)—i.e., drones—has emerged in the past 5 years. A significant challenge in implementing digitisation and computerisation is the high cost and long-term investment, particularly in small and medium-sized companies and in developing countries [25].

An RTK drone survey was used to quickly and homogeneously determine the amount of earthwork [40]. Data were collected through the survey and tracking was performed in BIM. While, in this study, the process worked well and the results were accurate, it was noted that the availability of data (3D IFC databases) and terrain obstacles such as water and vegetation might negatively impact the effectiveness of the method. Visual data collected by UAVs could be processed by artificial intelligence and machine learning using the YOLOv9 algorithm, not only for earthwork and volume calculation but also for crack detection [45,46].

Data collection is possible on the ground as well, using unmanned ground vehicles (UGVs). The pavement layer change classifier automatically measures pavement construction progress [47]. In the cited study, the UGV was equipped with a laser distance sensor, an accelerometer, a gyroscope, and a GPS sensor. The authors selected various scenarios and different sensors and algorithms to compare and select the best combination. The best combination's highest accuracy was 97.88%. This real-time automated monitoring can help companies to move away from tedious and time-consuming manual methods.

Digital twins were applied to road construction by integrating BIM, IoT, and intelligent compaction data into a virtual road model. It mirrors on-site compaction in real time for quality monitoring and visualisation [48]. This BIM-IoT-IC platform enables the continuous tracking of compaction performance and supports the dynamic rescheduling and optimisation of construction activities based on live quality feedback. There are ad-

ditional significant applications of digital twins and blockchain applications in supply chain management.

3.5. Other Important Considerations

Green road construction practices in Ghana were analysed [49] using a quantitative approach and structural equation modelling (SEM) to identify four key components: sustainable design, materials and resources, stormwater management, and energy efficiency. The results reveal stormwater management as the most significant component, followed by the use of locally sourced materials and energy efficiency measures, with sustainable design focusing on educating workers about environmental hazards. The findings provide a framework for stakeholders to implement sustainable road construction practices and contribute valuable empirical evidence to the literature on green infrastructure in developing countries.

4. Discussion

This comprehensive bibliometric review and analysis of recent scholarly articles identifies significant opportunities to enhance efficiency, sustainability, and cost-effectiveness in road construction. The majority of the top cited articles fall into this category (Table 2), and a large proportion of the keywords are included in the green (efficiency) and blue (durability, stabilisation) clusters (Figure 4). There is an evident trend toward adopting alternative, recycled, and bio-based materials across the literature. These materials help to reduce the overall carbon footprint by extending the lifecycles of roads, using locally sourced materials to minimise transportation-related emissions, and turning waste into valuable construction inputs. Such innovations not only conserve scarce natural resources but also promote environmentally responsible construction practices that align with global sustainability goals.

Digital transformation emerges as a critical enabler of improved road construction productivity. Another large proportion of the keywords fell into the red cluster (Figure 4). Technologies such as building information modelling (BIM), advanced scheduling models, and intelligent monitoring systems, including drones and unmanned ground vehicles (UGVs), hold the promise of productivity increases of 20–25%. Artificial intelligence (AI) applications, particularly in optimising fuel and resource consumption, integrate seamlessly with these digital tools, reinforcing sustainability while enhancing operational efficiency. However, the broad adoption of these technologies is often hindered by high upfront costs, a lack of standardisation, and challenges that are unique to specific geographic and organisational contexts. Small and medium enterprises (SMEs), especially in developing regions, face pronounced barriers that impede their ability to leverage these advancements, underscoring an urgent need for accessible, scalable solutions.

The management of road construction projects benefits substantially from integrated approaches combining lean principles, Industry 4.0, and system-of-systems methodologies, forming the yellow cluster of keywords (Figure 4). Enhanced planning, communication, and real-time analytics can reduce waste, improve machine coordination, and shorten project durations without increasing costs. The application of hybrid analytical models incorporating sustainability dimensions reveals that economic, social, and environmental factors can be harmonised for resilient and efficient road network development. Concurrently, innovations in materials science—from bio-rejuvenators and self-healing concrete to novel binders and composites—demonstrate potential to improve road durability, reduce maintenance cycles, and mitigate environmental impacts.

There is strong pressure on road construction companies to decrease costs. Digitisation makes control and decision-making more transparent, and a data-driven management

method can be leveraged and improved based on feedback from the sector. There is further pressure from growing sustainability targets regarding carbon dioxide emissions over the whole lifecycle of construction. We have developed a conceptual model (Figure 11) illustrating the feedback driven by both pressures on the sector driving innovation in materials [35,36,38]: bonding materials that require less heat to produce than pozzolan cement, locally available by-products (cheaper transportation, cheaper materials), and as improving management methods by leveraging IT technologies that help data-driven decision-making and provide better transparency [7–11]. In this model, cost-efficiency and sustainability targets drive distinct improvements that ultimately affect both. Innovative materials not only lead to greener production but also lower costs, while digitisation-driven efficiency leads to cleaner production (e.g., by saving fuel).

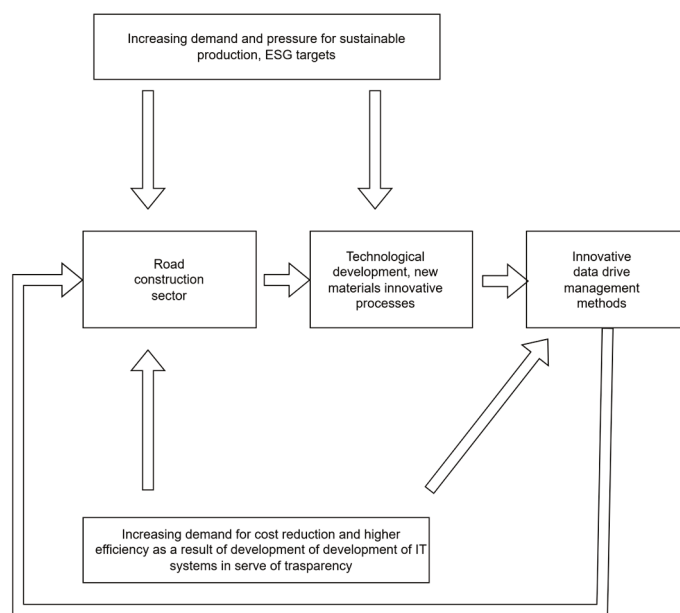


Figure 11. Conceptual framework linking environmental impact, digitalisation, and efficiency.

Despite these promising insights, critical implementation challenges persist. These include the need for the rigorous testing and standardisation of innovative materials and technologies to ensure reliability and safety and the development of comprehensive, adaptable guidelines that accommodate diverse project contexts. Additionally, greater international collaboration and knowledge exchange are necessary to bridge gaps between research and practice, particularly in regions where infrastructure development is rapidly expanding but access to cutting-edge technologies remains limited.

Future Research Agenda

Building on the findings, the future research agenda in road construction should encompass several prioritised dimensions.

Standardisation and Validation of Innovative Materials and Technologies: There is a clear need for robust, standardised testing protocols and performance benchmarks addressing the long-term behaviour of alternative materials, bio-based stabilisers, and digital construction technologies under diverse environmental and operational conditions.

Affordable Digital Solutions for SMEs and Developing Countries: Research should focus on developing cost-effective, modular, digital, and AI-driven tools tailored to the resource constraints of SMEs, emphasising ease of integration, scalability, and impacts on productivity and sustainability.

Cross-Disciplinary and International Collaboration: Encouraging collaborative, multi-institutional studies that bring together expertise from civil engineering, materials science, computer science, environmental science, and economics will foster holistic solutions. Particular attention is needed to include developing country perspectives and customise innovations accordingly. As illustrated in Figure 8, the corresponding authors' countries have a relatively low level of multi-country cooperation.

Lifecycle and Circular Economy Models: Advancing comprehensive lifecycle assessment methods and circular economy frameworks tailored to road construction materials and processes will enable the quantification of environmental impacts and identification of trade-offs, supporting decision-making for sustainable infrastructure.

Policy and Socioeconomic Impact Studies: Investigations into the wider economic, social, and regulatory effects of adopting new road construction paradigms—including urbanisation impacts, labour market influences, and legal frameworks—will help to align technological gains with societal benefits.

Smart Infrastructure and Resilience: Further exploration of smart road systems leveraging the Internet of Things (IoT), sensor networks, and real-time data analytics can enhance infrastructure resilience, maintenance, and user safety. Research should address the integration challenges and environmental footprints of such systems.

Adaptive Project Management Methodologies: There is an opportunity for research into innovative project management approaches that incorporate uncertainty, variability in material performance, and dynamic stakeholder needs, utilising advanced modelling and optimisation techniques.

By addressing these agenda items through targeted research efforts, the road construction field can accelerate its transformation toward more efficient, environmentally responsible, and economically viable infrastructure development globally.

Author Contributions: Conceptualization, B.B., Z.K.L., and K.N.; methodology, K.N.; software, K.N.; validation, B.B. and Z.K.L.; formal analysis, B.B., Z.K.L., and K.N.; investigation B.B. and K.N.; resources Z.K.L.; data curation, K.N.; writing—original draft preparation, B.B., Z.K.L., and K.N.; writing—review and editing, B.B., Z.K.L., and K.N.; visualization, K.N.; supervision B.B.; project administration, B.B.; funding acquisition, B.B., All authors have read and agreed to the published version of the manuscript.

Funding: Supported by the EKÖP-MATE/2025/26/D University Research Scholarship Programme of the Ministry for Culture and Innovation from the Resources of the National Research, Development and Innovation Fund.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding author.

Conflicts of Interest: The authors declare no conflict of interest.

References

1. Rathee, C.; Sadhukhan, S. Regional impact assessment of highways and policy Interventions: Lessons from Haryana, India. *Case Stud. Transp. Policy* **2025**, *20*, 101421. [CrossRef]
2. Tian, G.; Li, J. How Does Infrastructure Construction Affect Economic Development along the “Belt and Road”: By Promoting Growth or Improving Distribution? *Emerg. Mark. Financ. Trade* **2019**, *55*, 3332–3348. [CrossRef]
3. Pienaar, W. The extension of cost-benefit analysis with social analysis in the planning of public road construction projects: Suggestion in support of the creation of a developmental state. *Tydskr. Vir Geesteswet.* **2014**, *54*, 753–770.
4. Anish, C.; Kiruthiga, K.; Vinoth, S. Analysis of time delay and cost overrun in road construction. *Int. J. Innov. Technol. Explor. Eng.* **2019**, *8*, 901–907. [CrossRef]
5. De Winter, H.; Bassier, M.; De Geyter, S.; Vergauwen, M. Automatic calculation of volume changes in road construction. In *Optical 3D Metrology (O3DM)*; International Archives of the Photogrammetry, Remote Sensing and Spatial Information Sciences; Remondino, F., Shortis, M., Nuchter, A., Eds.; Copernicus Gesellschaft MbH: Gottingen, Germany, 2022; pp. 20–27. [CrossRef]

6. Axelsson, J.; Froberg, J.; Eriksson, P. Towards a System-of-Systems for Improved Road Construction Efficiency Using Lean and Industry 4.0. In Proceedings of the 2018 13th Annual Conference on System of Systems Engineering (SOSE), Paris, France, 19–22 June 2018; IEEE: New York, NY, USA, 2018; pp. 576–582.
7. Dagou, H.H.; Gurgun, A.P.; Koc, K.; Budayan, C. The Future of Construction: Integrating Innovative Technologies for Smarter Project Management. *Sustainability* **2025**, *17*, 4537. [CrossRef]
8. Chen, S.; Zeng, Y.; Majdi, A.; Salameh, A.A.; Alkhalifah, T.; Alturise, F.; Ali, H.E. Potential features of building information modelling for application of project management knowledge areas as advances modeling tools. *Adv. Eng. Softw.* **2023**, *176*, 103372. [CrossRef]
9. Matthei, J.; Klemm-Albert, K. Data driven planning and approval processes of public road construction projects. In *EUROSTRUCT 2023: European Association on Quality Control of Bridges and Structures: Digital Transformation in Sustainability*; Ernst & Sohn Verlag Architektur & Technische Wissenschaften GmbH & Co Kg: Berlin, Germany, 2023; Volume 6, pp. 365–371. [CrossRef]
10. Mirhosseini, A.F.; Pitera, K.; Odeck, J.; Welde, M. Sustainable Project Management: Reducing the Risk of Cost Inaccuracy Using a PLS-SEM Approach. *Sustainability* **2022**, *14*, 960. [CrossRef]
11. Lee, C.; Cha, M.; Kim, H.; Cho, H. Detailed Activity-Based Earthwork Scheduling Model to Aid during the Planning Stage of Road Construction. *Appl. Sci.* **2023**, *13*, 12902. [CrossRef]
12. Barati, K.; Shen, X. Modeling and Optimizing Fuel Usage of On-Road Construction Equipment. In *Construction Research Congress 2018: Sustainable Design and Construction and Education*; Wang, C., Harper, C., Lee, Y., Harris, R., Berryman, C., Eds.; American Society of Civil Engineers: New York, NY, USA, 2018; pp. 198–207.
13. Kotlobai, A.J.; Kotlobai, A.A.; Tamelo, V.F. Hydraulic Units for Driving Systems of Running Equipment in Road Construction Machinery. *Sci. Tech.* **2016**, *15*, 69–77. [CrossRef]
14. Singh, A.; Srivastava, A. Innovation needed to improve road construction and road infrastructure in the future in India. *IOP Conf. Ser. Earth Environ. Sci.* **2024**, *1326*, 012097. [CrossRef]
15. Wacker, B.; Jansen, D. Two years of APT program on the new test site duraBAST. In *Accelerated Pavement Testing to Transport Infrastructure Innovation: Proceedings of 6th APT Conference*; Springer International Publishing: Cham, Switzerland, 2020; pp. 32–39. [CrossRef]
16. Mhana, K.H.; Norhisham, S.B.; Katman, H.Y.B.; Yaseen, Z.M. Road urban planning sustainability based on remote sensing and satellite dataset: A review. *Heliyon* **2024**, *10*, e39567. [CrossRef] [PubMed]
17. Varma, S.; Jamrah, A.; Kutay, M.E.; Korkmaz, K.A.; Haider, S.W.; Buch, N. A framework based on engineering performance and sustainability to assess the use of new and recycled materials in pavements. *Road Mater. Pavement Des.* **2019**, *20*, 1844–1863. [CrossRef]
18. Stöckner, M.; Buttgerit, A.; Stöckner, U. Asset Management of Municipal Road Infrastructure. In *International Conference on Computing in Civil and Building Engineering*; Springer Nature: Cham, Switzerland, 2025; pp. 228–240. [CrossRef]
19. Besenbruth, I. Comparative Life cycle analysis of typical construction methods in road construction. *Betonw. Fert.-Tech. Plant Precast Technol.* **2025**, *91*, 27.
20. Shi, J.; Hu, Y.; Li, S.; Li, Z.; Zhang, X.; Mao, C. Simulation and analysis of road construction traffic flow in urban road networks. *Adv. Mech. Eng.* **2015**, *7*, 1687814015618176. [CrossRef]
21. Sukhinin, S.; Zilberova, I.; Novoselova, I.; Petrov, K.; Borovoy, V. Specifics of infrastructure construction under the conditions of urban. *BIO Web Conf.* **2024**, *138*, 04014. [CrossRef]
22. Ji, Z.; Huang, Y. The Impact of Highway Infrastructure on Population Mobility: Evidence from a Sample of 800 Counties and Districts (2000–2019) in China. *Sustainability* **2023**, *15*, 14834. [CrossRef]
23. Pompigna, A.; Mauro, R. Smart roads: A state of the art of highways innovations in the Smart Age. *Eng. Sci. Technol. Int. J.* **2022**, *25*, 100986. [CrossRef]
24. Alnadish, A.M.; Bangalore Ramu, M.; Kasim, N.; Alawag, A.M.; Baarimah, A.O. A Bibliometric Analysis and Review on Applications of Industrial By-Products in Asphalt Mixtures for Sustainable Road Construction. *Buildings* **2024**, *14*, 3240. [CrossRef]
25. Pentury, C.; Karsaman, R.H.; Rahman, H.; Rosmansyah, Y. Application of digitalization and computerization technology in road construction. *Autom. Constr.* **2025**, *171*, 106018. [CrossRef]
26. Pana, D.; Li, L.; Tian, X.; Wu, Y.; Cheng, N.; Yu, H. A review on lead slag generation, characteristics, and utilization. *Resour. Conserv. Recycl.* **2019**, *146*, 140–155. [CrossRef]
27. Xiao, Y.; Zhong, J.-L.; Zhang, Q.-F.; Xiang, X.; Huang, H. Exploring the coupling coordination and key factors between urbanization and land use efficiency in ecologically sensitive areas: A case study of the Loess Plateau, China. *Sustain. Cities Soc.* **2022**, *86*, 104148. [CrossRef]
28. Fang, Y.; Zhang, Z.; Yang, J.; Li, X. Comprehensive review on the application of bio-rejuvenator in the regeneration of waste asphalt materials. *Constr. Build. Mater.* **2021**, *295*, 123631. [CrossRef]

29. Bamigboye, G.O.; Bassey, D.E.; Olukanni, D.O.; Ngene, B.U.; Adegoke, D.; Odetoyan, A.O.; Kareem, M.A.; Enabulele, D.O.; Nworgu, A.T. Waste materials in highway applications: An overview on generation and utilization implications on sustainability. *J. Clean. Prod.* **2021**, *283*, 124581. [CrossRef]
30. Zentar, R.; Wang, H.; Wang, D. Comparative study of stabilization/solidification of dredged sediments with ordinary Portland cement and calcium sulfo-aluminate cement in the framework of valorization in road construction material. *Constr. Build. Mater.* **2021**, *279*, 122447. [CrossRef]
31. Mah, C.M.; Fujiwara, T.; Ho, C.S. Life cycle assessment and life cycle costing toward eco-efficiency concrete waste management in Malaysia. *J. Clean. Prod.* **2018**, *172*, 3415–3427. [CrossRef]
32. Zhao, Z.; Benzerzour, M.; Abriak, N.-E.; Damidot, D.; Courard, L.; Wang, D. Use of uncontaminated marine sediments in mortar and concrete by partial substitution of cement. *Cem. Concr. Compos.* **2018**, *93*, 155–162. [CrossRef]
33. Lee, S.; Chung, M.; Park, H.M.; Song, K.-I.; Chang, I. Xanthan Gum Biopolymer as Soil-Stabilization Binder for Road Construction Using Local Soil in Sri Lanka. *J. Mater. Civ. Eng.* **2019**, *31*, 06019012. [CrossRef]
34. Pecqueur, G.; Crignon, C.; Quénée, B. Behaviour of cement-treated MSWI bottom ash. *Waste Manag.* **2001**, *21*, 229–233. [CrossRef]
35. Mousi, J.B.; Kamwa, R.A.T.; Nemaleu, J.G.D.; Tome, S.; Gerard, M.; Etoh, M.-A.; Oyong, C.P.M.; Etame, J. Improvement of the bearing capacity of lateritic gravel by a geopolymer binder: Road construction in tropical countries. *Innov. Infrastruct. Solut.* **2024**, *9*, 242. [CrossRef]
36. Renjith, R.; Robert, D.; Fuller, A.; Setunge, S.; O'Donnell, B.; Nucifora, R. Enzyme based soil stabilization for unpaved road construction. In *6th International Conference of Euro Asia Civil Engineering Forum (EACEF 2017), Matec Web of Conferences*; Park, J.W., Lie, H.A., Hardjasaputra, H., Thayaalan, P., Eds.; E D P Sciences, Cedex A: Les Ulis, France, 2017; p. 01002. [CrossRef]
37. Belyaev, A.; Nesvetaev, G.; Mailyan, D. The Issues of Energy-Efficiency Increase of Three-Layer Reinforced Concrete Plate Constructions. In *International Scientific Conference Energy Management of Municipal Transportation Facilities and Transport, EMMFT 2017*; Murgul, V., Popovic, Z., Eds.; Springer International Publishing: Heidelberg, Germany, 2018. [CrossRef]
38. Vorotyneva, A.; Ovsiannikov, A.; Bolgov, V. Estimated Valuation of the Cost of Road Construction with the Use of Composite Cationic Bituminous Structures. In *International Science Conference Spbwosce-2016—Smart City*; Murgul, V., Ed.; E D P Sciences: Les Ulis, France, 2017; Volume 106, p. 08037. [CrossRef]
39. Lee, S.; Gonsalves, S. Inclusive Approach in Advancing Sustainability in Road Construction. In *International Conference on Transportation Geotechnics*; Springer Nature: Singapore, 2025; pp. 257–270. [CrossRef]
40. Dalai, B.; Kumar, A.; Landge, V. Sustainable Highway Infrastructure: Materials, Technologies, and Rating System Approach. In *Recent Advancements in Sustainable and Safe Transportation Infrastructure—Vol. 1. CTSEM 2024*; Srinivasan, V., Jain, U., Anjaneyulu, M.V.L.R., Parida, M., Eds.; Lecture Notes in Civil Engineering; Springer: Singapore, 2025; Volume 621. [CrossRef]
41. Foth, M.; Haichert, R.; Guenther, D.; Berthelot, C. Sustainability case study review of using recycled aggregate in road structure. In *Proceedings of the 2011 Conference and Exhibition of the Transportation Association of Canada. Transportation Successes: Let's Build on Them. 2011 Congress et Exhibition de l'Association des Transports du Canada. Les Succes en Transports: Une Tremplin vers l'Avenir*, Edmonton, AB, Canada, 11–14 September 2011.
42. Cherian, C.; Siddiqua, S. Engineering and environmental evaluation for utilization of recycled pulp mill fly ash as binder in sustainable road construction. *J. Clean. Prod.* **2021**, *298*, 126758. [CrossRef]
43. Mahmoudi, R.; Shetab-Boushehri, S.-N.; Hejazi, S.R.; Emrouznejad, A.; Rajabi, P. A hybrid egalitarian bargaining game-DEA and sustainable network design approach for evaluating, selecting and scheduling urban road construction projects. *Transp. Res. Part E-Logist. Transp. Rev.* **2019**, *130*, 161–183. [CrossRef]
44. Hou, B.; Mao, C.; Xu, L.; Sun, B.; Zhang, J. Evaluation Method of Municipal Road Construction Quality Based on Carbon Emissions using Genetic Algorithm. *Math. Probl. Eng.* **2022**, *2022*, 3434683. [CrossRef]
45. Iqbal, M.H.; Iqbal, M.J.; Saleem, T. Road layer detection and volume calculation using UAV Technologies and Artificial Intelligence. *Eng. Res. Express* **2025**, *7*, 015111. [CrossRef]
46. Nguyen, N.H.; Nguyen, T.-H.T.; Bui, L.G.; Ly, H.-B. Enhancing concrete structure maintenance through automated crack detection: A computer vision approach. *J. Sci. Transp. Technol.* **2024**, *4*, 11–23. [CrossRef]
47. Patel, T.; Guo, B.H.W.; van der Walt, J.D.; Zou, Y. Effective Motion Sensors and Deep Learning Techniques for Unmanned Ground Vehicle (UGV)-Based Automated Pavement Layer Change Detection in Road Construction. *Buildings* **2023**, *13*, 5. [CrossRef]
48. Han, T.; Ma, T.; Fang, Z.; Zhang, Y.; Han, C. A BIM-IoT and intelligent compaction integrated framework for advanced road compaction quality monitoring and management. *Comput. Electr. Eng.* **2022**, *100*, 107981. [CrossRef]
49. Kissi, E.; Babon-Ayeng, P.; Aigbavboa, C.; Duah, D.; Danquah-Smith, E.; Tannor, R.A. Examining green road construction components: The case of Ghana. *Built Environ. Proj. Asset Manag.* **2023**, *13*, 682–699. [CrossRef]

Disclaimer/Publisher's Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.

MDPI AG
Grosspeteranlage 5
4052 Basel
Switzerland
Tel.: +41 61 683 77 34

Applied Sciences Editorial Office
E-mail: appls@mdpi.com
www.mdpi.com/journal/appls



Disclaimer/Publisher's Note: The title and front matter of this reprint are at the discretion of the Guest Editors. The publisher is not responsible for their content or any associated concerns. The statements, opinions and data contained in all individual articles are solely those of the individual Editors and contributors and not of MDPI. MDPI disclaims responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.



Academic Open
Access Publishing

mdpi.com

ISBN 978-3-7258-8670-8