

Optimization of Mechanical Properties and Quality Index of Perlite Lightweight Concrete Reinforced with Microsilica and Glass Wool Fibers Using Response Surface Methodology

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ABSTRACT

Perlite lightweight concrete has attracted considerable attention because of its low density and favorable thermal performance. However, its relatively low mechanical strength and brittle behavior still limit its structural applications. This study investigates the combined effects of microsilica content (7.5–12.5%), glass wool fiber dosage (0–1.5 vol%), and fiber length (5–50 mm) on the mechanical properties and quality index of perlite lightweight concrete. A total of 30 concrete mixtures were designed and experimentally tested under compression, and the obtained data were subsequently analyzed using response surface methodology (RSM) to evaluate factor interactions and determine the optimum mixture proportions. Compressive strength tests were conducted on all mixtures, whereas stress-strain behavior was evaluated for 14 representative mixtures selected to characterize the deformation response. Overall, the simultaneous incorporation of microsilica and glass wool fibers improved the compressive strength, ductility, and energy absorption capacity of perlite lightweight concrete. The experimental results demonstrated that the highest compressive strength of 40.0 MPa was achieved for the mixture containing 10% microsilica and 0.5% glass wool fibers with a length of 27.5 mm. In contrast, the RSM-based multi-objective optimization identified a different optimum mixture containing approximately 9.4% microsilica and 0.9% glass wool fibers, which yielded the maximum quality index (R/ρ), defined as the compressive strength-to-density ratio, representing the optimum balance between mechanical performance and density. The novelty of this study lies in integrating experimental investigation with RSM-based multi-objective optimization to distinguish between the optimum compressive strength obtained experimentally and the mathematically optimized quality index, thereby providing a practical framework for optimizing both strength and strength-to-density efficiency of structural lightweight concrete.

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1. Introduction

In recent decades, the development of high-performance lightweight concrete has received considerable attention as an effective approach for reducing structural dead load and improving thermal performance [1–3]. However, reducing density is often associated with a decline in mechanical strength and durability; therefore, the incorporation of pozzolanic materials and reinforcing fibers has been widely proposed to compensate for these deficiencies. Microsilica, as a highly reactive pozzolanic material, enhances the

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cement paste microstructure, reduces porosity, and improves the interfacial transition zone (ITZ) between paste and aggregates, thereby increasing concrete strength and durability. In addition, glass wool fibers improve post-peak behavior, ductility, and energy absorption capacity by controlling microcrack propagation and transferring stresses across crack surfaces [4, 5].

Several studies have investigated the influence of different additives and fibers on concrete performance. Zeyad et al. [6] demonstrated that the use of heavyweight aggregates together with steel, basalt, and bent-fiber reinforcements enhanced both mechanical properties and gamma-ray shielding performance, although their primary focus was radiation protection rather than lightweight concrete optimization. Abdelsalam et al. [7] reported that natural fiber ropes improved crack control and ductility in reinforced concrete beams, although the ultimate load-carrying capacity decreased. Similarly, Dziomdziora and Smarzewski [8] showed that hybrid fiber systems can enhance the mechanical performance and post-cracking behavior of concrete; however, achieving uniform fiber dispersion remains a major challenge.

Standard-based investigations indicate that ASTM (American Society for Testing and Materials) standards do not provide a specific criterion for determining stress reduction levels in energy absorption calculations, and most studies adopt a range between 20% and 80% of peak stress. According to Deng et al. [9], a 50% reduction from peak stress represents a stable and reproducible criterion for evaluating absorbed energy.

Altalib et al. [10], Fallahatabar Shiade and Tavakoli [11], Akbarpour et al. [12], and Tavakoli [13] investigated the effects of pozzolanic materials and different fiber types on the mechanical properties and durability of concrete. Their findings demonstrated that pozzolanic additives significantly improve flexural strength, energy absorption capacity, and concrete performance under elevated temperatures. In addition, these studies evaluated the applicability of intelligent predictive methods and non-destructive techniques, such as ultrasonic pulse velocity, for estimating the properties of fiber-reinforced self-compacting concrete. Despite these advances, previous investigations have generally focused on individual parameters or specific fiber systems rather than simultaneously optimizing microsilica dosage, glass wool fiber content, and fiber length.

Recent studies [14–18] have confirmed this limitation and proposed various approaches to improve the mechanical performance of lightweight concrete. In this regard, microsilica and reinforcing fibers are considered among the most effective approaches for improving post-peak resistance, fracture strain, and energy absorption capacity. Most previous studies have investigated either supplementary cementitious materials or reinforcing fibers individually, whereas limited attention has been paid to their combined interaction in perlite lightweight concrete. Furthermore, previous investigations have rarely considered the simultaneous optimization of microsilica dosage, glass wool fiber content, and fiber length using a statistical multi-objective approach. In addition, distinguishing between the experimentally obtained optimum mechanical performance and the mathematically optimized quality index has not been adequately addressed in the existing literature.

Accordingly, the present study investigates the combined effects of microsilica dosage, glass wool fiber content, and fiber length on the compressive strength, stress-strain behavior, energy absorption capacity, and quality index of perlite lightweight concrete. Response Surface Methodology (RSM) was employed to quantify the interaction between variables and to perform multi-objective optimization. Unlike previous studies, this research distinguishes between the experimentally measured optimum compressive strength and the RSM-predicted optimum quality index, thereby providing a practical framework for proportioning structural lightweight concrete with improved mechanical efficiency.

2. Materials and methods

2.1. Preparation of materials

Microsilica was used as a supplementary cementitious material because of its well-established pozzolanic activity and its beneficial effects on the microstructure and mechanical performance of concrete [19]. Glass wool fibers were incorporated as reinforcement because previous studies have demonstrated their beneficial effects on crack control and the mechanical performance of cementitious composites [20–22]. In the present study, perlite was selected as the lightweight aggregate due to its low density, favorable thermal insulation properties, compatibility with cement paste, and wide availability, making it suitable for producing structural lightweight concrete with acceptable mechanical performance. In the present study, perlite was selected as the lightweight aggregate due to its low density, favorable thermal insulation properties, compatibility with cement paste, and wide availability, making it suitable for producing structural lightweight concrete with acceptable mechanical performance.

For specimen preparation, type II Portland cement, natural coarse aggregate, and lightweight perlite aggregate were used. Lightweight perlite fine aggregate complied with ASTM C330/C330M-23 [23], while coarse aggregates satisfied ASTM C33/C33M-18 [24] requirements. Aggregate gradation was modified according to the standard limits, and the amounts of perlite and coarse aggregate were adjusted proportionally to maintain a constant total mixture volume with varying microsilica replacement levels. Cylindrical specimens with dimensions of 105 mm × 200 mm were then prepared for compressive strength testing in accordance with ASTM C39 (Fig. 1).

As illustrated in Fig. 2, fine and coarse aggregates were first sieved separately. Dry constituents, including cement, perlite, microsilica, coarse aggregate, and glass wool fibers, were then weighed and mixed, followed by the gradual addition of water and superplasticizer.

2.2. Concrete mixture proportions

The concrete mixture proportions were determined based on ASTM C330/C330M-23 [23], ASTM C567-05a [25], ASTM C496-96 [26], ASTM C469-02 [27], ASTM C39/C39M-21 [28], and ACI 211.2-98 [29] recommendations. The final mixture consisted of approximately 900 kg/m³ coarse aggregate, 250 kg/m³ lightweight fine aggregate, 510 kg/m³ cement, 170 kg/m³ water, and 10 kg/m³ superplasticizer, corresponding to a water-to-cement ratio of 0.335.

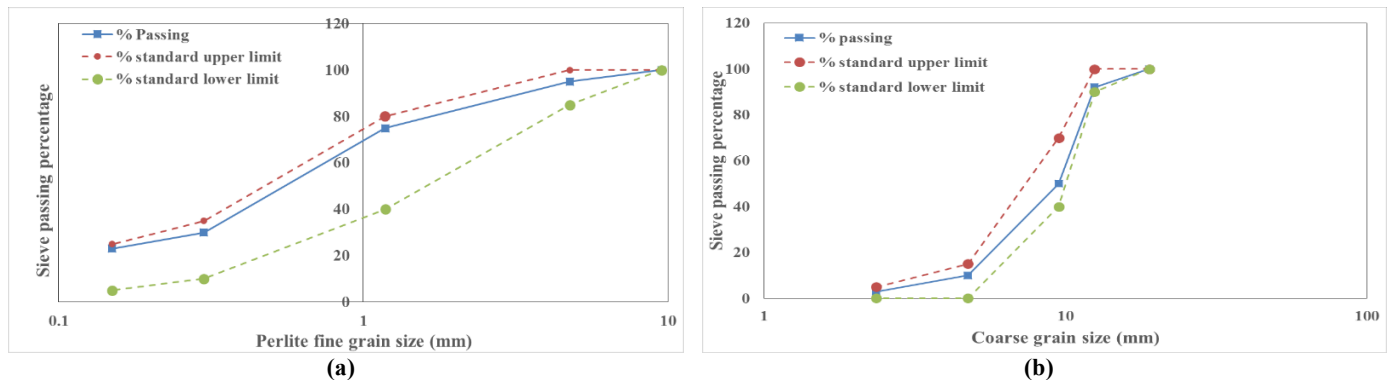


Fig. 1. Modified aggregate gradation curves used in the preparation of lightweight perlite concrete: (a) gradation of perlite fine aggregate; and (b) gradation of coarse aggregate.



Fig. 2. Aggregate separation and incorporation of glass wool fibers.

The selected ranges for glass wool fiber content (0-1.5% by volume) and fiber length (5-50 mm) were determined based on preliminary experiments and previous studies to cover practical ranges for structural lightweight concrete while preventing fiber agglomeration during mixing [30, 31]. The fiber length of 0.17 cm was initially considered in the experimental design; however, it was excluded from the final design space due to its statistically insignificant effect in the ANOVA analysis and its negligible influence on the mechanical responses. Microsilica was incorporated at replacement levels of 7.5%, 10%, and 12.5% by weight of cement. Glass wool fibers were added at volumetric fractions of 0%, 0.5%, 1%, and 1.5%, with lengths of 5, 27.5, and 50 mm.

To ensure uniform fiber dispersion, dry materials including cement, saturated surface-dry (SSD) perlite, microsilica, aggregate, and fibers were initially mixed for 10 min. Water and superplasticizer were then gradually introduced, and mixing continued for an additional 15 min to prevent fiber balling. Fresh concrete was cast into cylindrical molds with dimensions of 105 mm in diameter and 200 mm in height and compacted in three layers to minimize entrapped air. Specimens were initially cured under moist laboratory conditions for 48 h and subsequently demolded. Thereafter, all specimens were water-cured for 42 days to ensure stabilization of the pozzolanic reaction of microsilica [30, 31]. Before testing, specimen dimensions and weights were measured, and their ends were capped with molten sulfur to provide smooth loading surfaces.

The experimental program was designed independently of the RSM analysis. After completing all laboratory tests, the experimental results were imported into Design-Expert software (Version 13) [32] for regression modeling, statistical analysis (ANOVA), response surface generation, and multi-objective optimization.

Although slump was not quantitatively measured in this study, the workability of all mixtures was visually monitored during mixing and casting. Increasing the glass wool fiber content tended to reduce workability and required careful mixing to maintain uniform fiber dispersion and prevent fiber balling. The use of a superplasticizer and prolonged mixing ensured satisfactory casting quality for all mixtures.

2.3. Compressive strength test

All mechanical tests were performed at the age of 42 days. Compressive strength tests were conducted on cylindrical specimens in accordance with ASTM C39 [28], since cylindrical samples provide more uniform behavior than cubic specimens in lightweight concrete evaluation. Loading was applied using a 50-ton capacity compression testing machine at a constant displacement rate of 0.13 mm/min following ASTM C469 [27], corresponding to an average stress rate of approximately 0.25 MPa/s. The maximum failure load of each specimen was recorded.

Furthermore, two Linear Variable Differential Transformers (LVDTs) were employed to measure axial displacement for stress-strain curve acquisition (Fig. 3).



Fig. 3. Experimental setup for the compressive strength test.

Subsequently, load-displacement data were converted into stress-strain values, and the corresponding stress-strain curves were plotted. For each of the 30 mixture proportions, three replicate cylindrical specimens were prepared and tested. The reported compressive strength, density, and quality index values represent the average of the three measurements. Representative failure patterns of the specimens are illustrated in Fig. 4.



Fig. 4. Representative failure patterns of lightweight perlite concrete specimens under compression.

3. Results and discussion

3.1. Stress-strain test

Among the 30 experimentally prepared concrete mixtures, 14 representative mixtures were selected for stress-strain evaluation. These mixtures were selected to represent the control mixture, mixtures with the highest compressive strength, mixtures exhibiting the highest ductility, and mixtures located near the boundaries of the investigated experimental domain. The selected 14 mixtures were used only for detailed stress-strain analysis, whereas the compressive strength results of all 30 mixtures are presented in Section 3.2. The corresponding stress-strain results are summarized in Table 1. Each value reported in Table 1 represents the average of three identical cylindrical specimens tested under the same loading conditions.

Table 1. Specifications and stress-strain test results of the specimens.

Specimen ID	Glass wool fiber content (%)	Glass wool fiber length (cm)	Microsilica content (%)	42-day compressive strength (MPa)	Strain at maximum stress (mm/mm)
1	0	-	7.5	36.79	0.00198
2	0	-	10	37.21	0.00195
3	0	-	12.5	33.43	0.00192
4	1	0.5	12.5	33.73	0.00229
5	0.5	2.75	10	39.99	0.00211
6	0.5	5	12.5	32.85	0.00218
7	0.5	0.5	7.5	38.88	0.00203
8	0.5	5	10	39.52	0.0022
9	1	5	10	39.51	0.00248
10	1	0.5	7.5	36.19	0.00236
11	1	2.75	12.5	33.52	0.00233
12	1.5	0.5	12.5	30.89	0.00256
13	1.5	2.75	10	38.00	0.00271
14	1.5	5	7.5	34.33	0.00287

The experimental results indicate that the stress-strain response of lightweight concrete is governed by the combined influence of microsilica dosage, glass wool fiber content, and fiber length. While microsilica primarily affected the peak stress and elastic response through matrix densification, glass wool fibers mainly influenced the post-peak deformation behavior by delaying crack propagation and increasing strain capacity. The following sections discuss the contribution of each parameter separately.

3.1.1. Effect of microsilica content

Increasing the microsilica content from 7.5% to 10% enhanced the compressive strength of lightweight concrete from approximately 36.8 to 37.2 MPa, indicating a moderate yet beneficial improvement in mechanical performance. This enhancement can mainly be attributed to the filler effect of microsilica particles, reduction in pore volume, and strengthening of the interfacial transition zone (ITZ) due to the pozzolanic reaction. However, further increasing the microsilica content to 12.5% resulted in excessive matrix densification and reduced stress redistribution capability within the cement paste. Consequently, the increase in compressive strength was no longer proportional to the increase in microsilica content, while the strain at peak stress tended to decrease, indicating a transition toward a relatively more brittle mechanical response. These observations are consistent with previous studies [33–37], which reported that microsilica replacement levels of approximately 8–10% generally provide the optimum balance between matrix densification and mechanical performance. Beyond this range, the increased specific surface area raises water demand and may reduce particle dispersion efficiency, resulting in only marginal or even negative improvements in mechanical properties.

3.1.2. Effect of glass wool fiber content

Increasing the glass wool fiber content led to higher ultimate strain values and improved ductility of lightweight concrete. Compared with the control mixture, specimens containing 1.5% glass wool fibers exhibited an approximately 45% increase in peak strain, confirming the substantial contribution of fibers to deformation capacity. The fibers acted as crack-bridging elements, absorbing part of the stress concentration energy and delaying rapid crack propagation. As a result, the concrete retained part of its load-carrying capacity after crack initiation and exhibited a more ductile response. This behavior is mainly associated with stress transfer through the fibers and improved stress distribution within the cementitious matrix. According to recent studies [38–43], incorporating Glass wool fibers within an optimum volumetric range of approximately 1–1.5% can significantly improve ultimate strain, energy absorption capacity, and ductility of lightweight concrete. Nevertheless, excessive fiber content, particularly beyond 1.5%, may lead to fiber agglomeration, reduced workability, weaker bonding, and formation of weak zones within the matrix. The present experimental results demonstrated that a fiber content of 1.5% produced the highest deformation capacity among the investigated mixtures. However, compressive strength did not increase proportionally because excessive fiber content tends to reduce workability and matrix uniformity. Therefore, the optimum fiber dosage depends on the selected performance criterion.

3.1.3. Effect of glass wool fiber length

Experimental observations indicated that increasing the glass wool fiber length from 5 to 50 mm enhanced the ultimate strain and energy absorption capacity of lightweight concrete. Longer fibers, especially 50 mm fibers, provided more effective mechanical interlocking with the cementitious matrix and improved crack-bridging action, thereby preventing sudden crack propagation and producing a smoother post-peak response. Through stress transfer along crack paths, the fibers redistributed stresses and enhanced the ductility of the concrete. In contrast, shorter fibers were mainly effective in controlling initial microcracks but had a more limited influence on the overall post-cracking behavior. Previous investigations [44–47] have also reported that increasing fiber length improves fiber-matrix bonding, fracture strain, and energy absorption capacity in lightweight and high-performance concretes, although excessively long fibers may lead to non-uniform distribution and localized fiber clustering. Although fiber length was treated as a numerical factor in the RSM analysis, the experimental program included only three discrete fiber lengths (0.5, 2.75, and 5 cm). Therefore, intermediate values predicted by the response surface model should be interpreted as mathematical interpolations within the investigated domain rather than experimentally validated optima. Additional experimental verification using intermediate fiber lengths is recommended in future studies to further assess the robustness and generalizability of the proposed model.

It should be emphasized that the 50 mm fibers exhibited the highest ultimate strain and energy absorption capacity, whereas the maximum quality index (R/ρ) was achieved with the 5 mm fibers. Accordingly, the optimum fiber length is response-dependent, and no single fiber length can be regarded as universally optimal for all mechanical performance indicators. Longer fibers are preferable for improving post-peak behavior and energy absorption, whereas shorter fibers provide higher strength-to-density efficiency when the quality index is adopted as the optimization criterion.

3.1.4. Elastic modulus and energy absorption

Fig. 5 presents representative stress-strain curves of lightweight concrete containing different glass wool fiber volume fractions (0–1.5%). The curves illustrate the influence of fiber reinforcement on both the pre-peak elastic response and the post-peak deformation behavior.

In the control mixtures (Mixes 1–3), which contain no glass wool fibers, the stress-strain curves exhibit a steep descending branch immediately after the peak stress, indicating limited post-peak deformation capacity and a relatively brittle mechanical response. In contrast, with increasing fiber content, the descending branch of the curve becomes more gradual, reflecting a more ductile and

progressive failure response. For example, the ultimate strain increased from approximately 0.00192–0.00198 in the control mixtures to 0.00271–0.00287 in the mixtures containing 1.5% glass wool fibers, corresponding to an increase of approximately 40–50%. This improvement demonstrates the effectiveness of glass wool fibers in enhancing the deformation capacity of lightweight concrete through crack-bridging and stress redistribution mechanisms. These observations indicate that the incorporation of microsilica and glass wool fibers significantly improves the post-peak deformation behavior of the investigated lightweight concrete mixtures. However, the applicability of these mixtures to specific structural members should be evaluated considering the design requirements and loading conditions of the intended application.

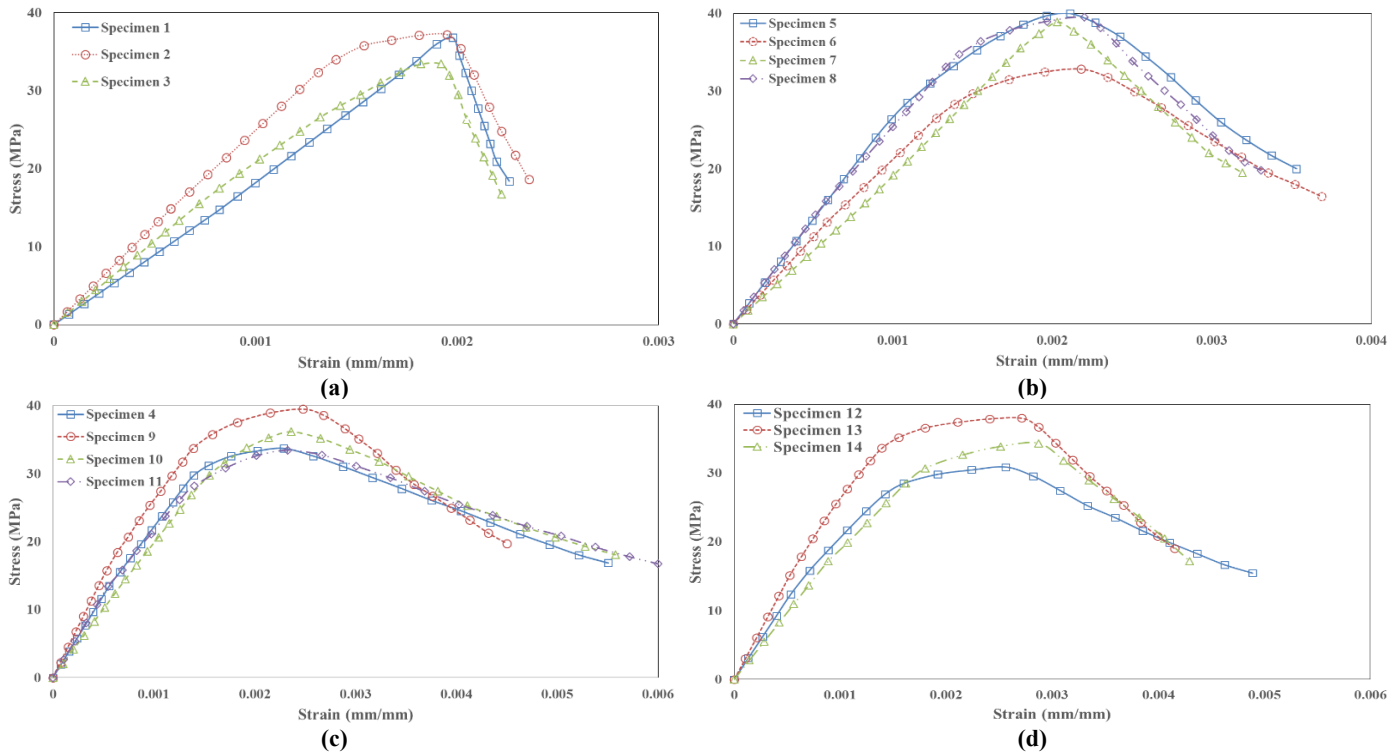


Fig. 5. Stress-strain behavior of lightweight perlite concrete containing different glass wool fiber contents: (a) control mixture without glass wool fibers; (b) mixture containing 0.5% glass wool fibers; (c) mixture containing 1.0% glass wool fibers; and (d) mixture containing 1.5% glass wool fibers.

For a fair comparison of energy absorption capacity among mixtures with different ultimate strains, the absorbed energy was calculated as the area under the stress-strain curve up to 50% of the peak stress. This approach, widely adopted in similar studies, enables consistent comparison of post-peak behavior. Based on the results presented in Table 2, the absorbed energy increased from approximately 60–78 kJ/m³ in the control mixtures to values as high as 194 kJ/m³ in the fiber-reinforced mixtures, representing an improvement of more than 150% under the investigated conditions.

Table 2. Modulus of elasticity and absorbed energy of the specimens.

Specimen ID	Modulus of elasticity (GPa)	Energy absorption up to 50% of peak stress (kJ/m ³)
1	17.91	64.63
2	25.64	77.55
3	21.52	60.63
4	24.12	178.93
5	26.93	129.27
6	22.36	113.18
7	18.79	104.73
8	27.18	121.13
9	29.73	170.53
10	20.08	188.10
11	24.58	194.13
12	23.22	145.91
13	28.89	154.37
14	19.49	137.87

The elastic modulus of each mix was determined according to ASTM C469-02 [27] using Eq. 1:

$$E = \frac{\sigma_2 - \sigma_1}{\varepsilon_2 - \varepsilon_1} \quad (1)$$

where σ_1 is the stress corresponding to a strain value of 0.00005, σ_2 is the stress corresponding to 0.4 ultimate strength, ε_1 is equal to 0.00005, and ε_2 are the strains corresponding to the stress σ_2 .

The results presented in Table 2 indicate that the elastic modulus and energy absorption capacity do not necessarily follow identical trends. While the elastic modulus mainly reflects the stiffness of the cementitious matrix before cracking, the absorbed energy is governed predominantly by the post-peak crack propagation process and the ability of glass wool fibers to bridge developing cracks. Consequently, mixtures with comparable elastic modulus values may exhibit substantially different energy absorption capacities. In this study, the effects of glass wool fiber content and length, as well as microsilica content, on the elastic modulus and energy absorption capacity of lightweight concrete were investigated. The results indicated that incorporating approximately 1% glass wool fibers produced the highest elastic modulus (29.73 GPa), consistent with previous studies [48], whereas the highest absorbed energy (194.13 kJ/m³) was obtained for another mixture containing the same fiber dosage but different microsilica content and fiber length. These findings demonstrate that the optimum mixture depends on the selected mechanical performance indicator rather than on a single response parameter. Increasing the fiber content to 1.5% resulted in a slight reduction in performance due to fiber agglomeration and reduced matrix homogeneity [49]. The investigation of fiber length revealed that the maximum energy absorption capacity was achieved at a fiber length of 2.75 cm, as this length provides more effective post-cracking reinforcement compared to the initial elastic response stage [50]. Increasing the microsilica content up to 10% enhanced both the elastic modulus and energy absorption, which is attributed to pore refinement, densification of the cement paste, and improved interfacial bonding [51]. However, further increase to 12.5% resulted in a reduction in performance, mainly due to higher water demand, non-uniform particle dispersion, and the formation of additional microcracks. The results also indicated that a direct linear relationship does not always exist between elastic modulus and energy absorption capacity. In some cases, specimens with lower modulus exhibited higher post-peak energy absorption, which is governed by the combined effect of fiber content, fiber length, and microsilica content [52]. Two failure modes were observed: brittle failure characterized by a sudden drop in the stress-strain curve, and ductile failure with a gradual post-peak decline. Overall, the experimental observations confirm that microsilica mainly enhances the stiffness and strength of the cementitious matrix through pore refinement and interfacial transition zone (ITZ) densification, whereas glass wool fibers primarily improve post-peak deformation behavior through crack-bridging mechanisms. The simultaneous incorporation of both materials therefore provides a balanced improvement in stiffness, strength, ductility, and energy absorption, explaining the superior overall mechanical efficiency observed in the optimized mixtures [49].

3.1.5. Microstructural analysis (SEM)

To investigate the influence of microsilica and glass wool fibers on the microstructure of lightweight perlite concrete, selected specimens were examined using scanning electron microscopy (SEM). SEM images at magnifications of 1000 $\times$ and 3000 $\times$ are presented in Fig. 6.

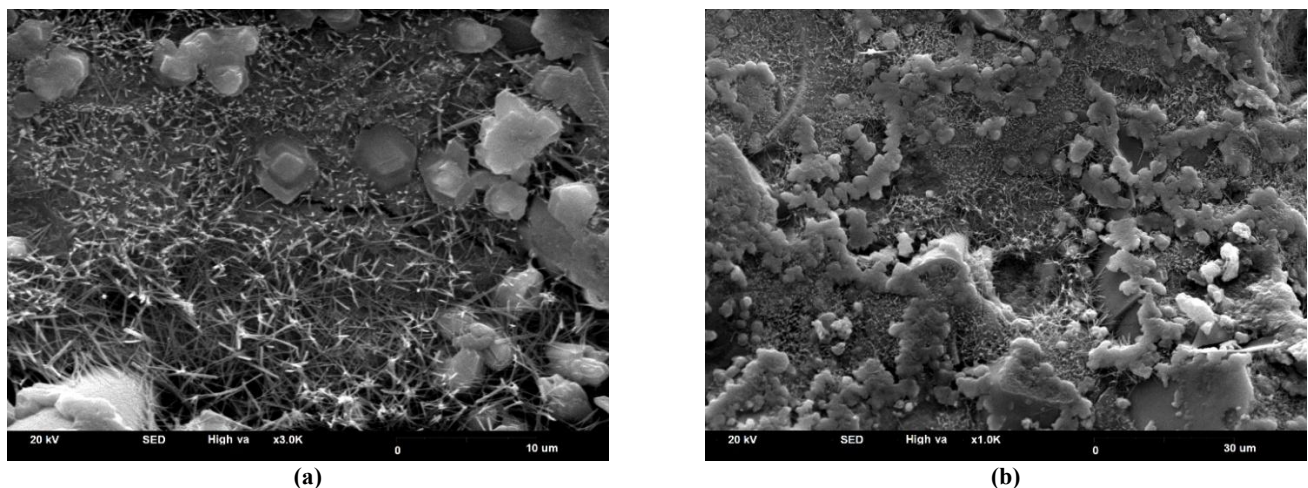


Fig. 6. SEM micrographs of lightweight perlite concrete containing microsilica and glass wool fibers: (a) microstructure at 1000 $\times$ magnification; and (b) microstructure at 3000 $\times$ magnification, illustrating the densified cementitious matrix and fiber-matrix interaction.

The microstructural observations revealed that the incorporation of microsilica contributed to matrix densification and a relative reduction in porosity within the lightweight concrete structure. The observed matrix densification can be attributed to the pozzolanic reaction between microsilica and calcium hydroxide released during cement hydration, producing additional calcium silicate hydrate (C-S-H) gel. This secondary hydration product refines the pore structure, strengthens the interfacial transition zone (ITZ), and contributes to the improvement in compressive strength observed in the experimental results. Due to its ultrafine particles and pozzolanic activity, microsilica filled microvoids and improved the uniformity and compactness of the cementitious matrix. In several regions, hydration products with needle-like and network-shaped morphologies were observed, indicating the formation of hydrated phases and improvement of the interfacial transition zone (ITZ) between the cement paste and aggregates. Higher magnification images further demonstrated that the glass wool fibers were well embedded within the cementitious matrix and acted as bridging elements across microcracks in some regions. The effective mechanical anchorage of the fibers within the cementitious matrix enabled efficient crack-bridging during loading, thereby delaying crack opening and reducing stress concentration at the crack tip. This mechanism explains the experimentally observed increase in ductility and post-peak energy absorption. Moreover,

the relatively good adhesion between the fibers and the surrounding matrix suggests an effective fiber-matrix interaction. The SEM observations confirm that the simultaneous incorporation of microsilica and glass wool fibers improved both matrix densification and crack-control mechanisms, leading to enhanced mechanical performance of lightweight perlite concrete. These findings are in good agreement with the compressive strength results, stress-strain behavior, and quality index analysis.

3.2. Compressive strength test

Compressive strength was experimentally determined for all 30 concrete mixtures after 42 days of water curing. For each mixture, three cylindrical specimens were tested under identical loading conditions, and the average compressive strength was used for subsequent statistical analysis and RSM modeling.

To evaluate the mechanical efficiency of lightweight concrete, a quality index (R/ρ) was adopted. This parameter is defined as the ratio of the 42-day compressive strength (R_{42} , MPa) to the concrete density (ρ , kg/m³). Although not dimensionless, this index provides a practical measure of strength achieved per unit density and is therefore useful for comparing structural lightweight concretes with different densities. Higher R/ρ values indicate greater mechanical efficiency. In this study, the quality index of lightweight concrete was defined as Eq. 2.

$$\text{Quality index} = \left(\frac{R_{42}}{\rho} \right) \times 100 \quad (2)$$

where R_{42} is the 42-day compressive strength and ρ is the concrete density. Unlike compressive strength, which evaluates only the load-bearing capacity, the quality index simultaneously considers strength and self-weight. Therefore, mixtures with the highest compressive strength do not necessarily exhibit the highest quality index. The complete experimental matrix together with the measured density, compressive strength, and calculated quality index of all 30 mixtures is presented in Table 3. All values correspond to experimentally measured data rather than RSM predictions.

Table 3. Specimen details and compressive strength test results.

Specimen ID	Glass wool fiber content (%)	Glass wool fiber length (cm)	Microsilica content (%)	42-day compressive strength (MPa)	Density (kg/m ³)	Quality index
1	0	-	7.5	36.79	1848	1.99
2	0	-	10	37.21	1842	2.02
3	0	-	12.5	33.43	1839	1.82
4	0.5	0.5	7.5	38.88	1829	2.13
5	0.5	0.5	10	39.91	1825	2.19
6	0.5	0.5	12.5	33.74	1820	1.85
7	0.5	2.75	7.5	36.49	1821	2.00
8	0.5	2.75	10	39.99	1818	2.20
9	0.5	2.75	12.5	33.38	1815	1.84
10	0.5	5	7.5	36.16	1807	2.00
11	0.5	5	10	39.52	1806	2.19
12	0.5	5	12.5	32.85	1800	1.83
13	1	0.5	7.5	36.19	1794	2.02
14	1	0.5	10	39.87	1787	2.23
15	1	0.5	12.5	33.73	1765	1.91
16	1	2.75	7.5	36.03	1789	2.01
17	1	2.75	10	39.65	1781	2.23
18	1	2.75	12.5	33.50	1763	1.90
19	1	5	7.5	35.76	1781	2.01
20	1	5	10	39.51	1779	2.22
21	1	5	12.5	31.70	1760	1.80
22	1.5	0.5	7.5	34.83	1751	1.99
23	1.5	0.5	10	38.22	1739	2.20
24	1.5	0.5	12.5	30.89	1724	1.79
25	1.5	2.75	7.5	35.18	1747	2.01
26	1.5	2.75	10	38.00	1733	2.19
27	1.5	2.75	12.5	30.19	1720	1.75
28	1.5	5	7.5	34.33	1740	1.97
29	1.5	5	10	37.79	1728	2.19
30	1.5	5	12.5	29.65	1718	1.73

The experimental results indicate that the highest compressive strength (40.0 MPa) was obtained for the mixture containing 10% microsilica, 0.5% glass wool fibers, and a fiber length of 27.5 mm. However, this mixture did not provide the highest quality index because its density was higher than that of several other mixtures. Consequently, the optimum mixture depends on the selected optimization criterion. Fig. 7 summarizes the experimentally measured variation of the quality index with respect to glass wool fiber dosage, microsilica content, and fiber length. These plots are presented solely to illustrate the experimental trends before statistical modeling.

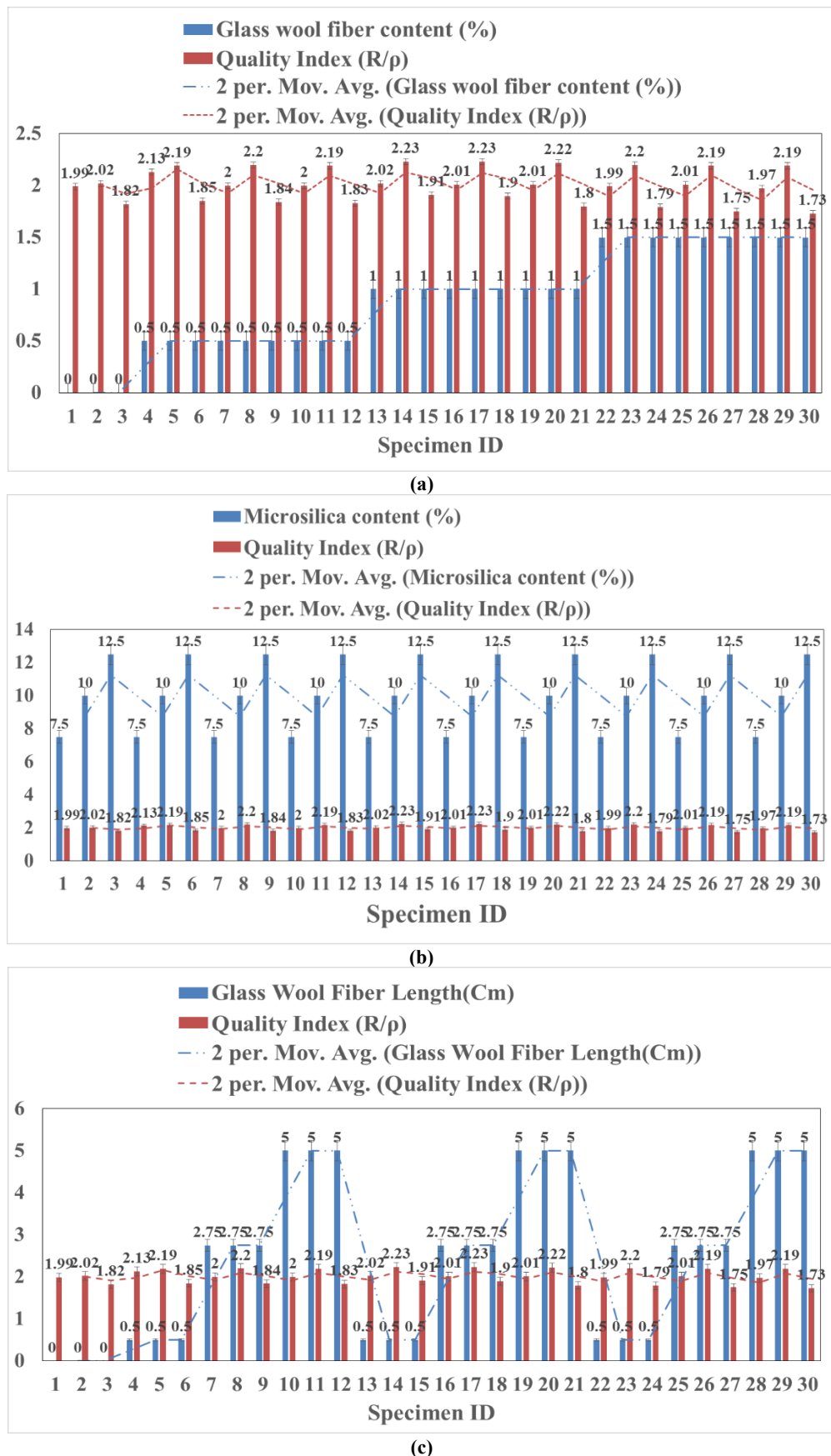


Fig. 7. Effects of the investigated variables on the quality index (R/ρ): (a) influence of glass wool fiber content; (b) influence of microsilica content; and (c) influence of glass wool fiber length.

Following the experimental investigation, the measured responses were imported into Design-Expert® (Version 13) [32] for statistical evaluation using Response Surface Methodology (RSM). The software was employed only for regression analysis and multi-objective optimization of the experimentally obtained results. To assess the effects of individual factors and their interactions on the response, analysis of variance (ANOVA) and regression modeling were employed. These methods enable the identification of statistically significant relationships between the input variables and the quality index.

3.2.1. Software data analysis

The experimentally measured quality index values listed in Table 3 were analyzed using Design-Expert® software (Version 13) [32]. Glass wool fiber content (factor A), microsilica content (factor B), and fiber length (factor C) were treated as independent variables, whereas the quality index (R/ρ) was considered the response variable.

In the box plots (Fig. 8), the median line and the first and third quartiles (25–75%) represent the distribution of the response data. The results indicated that increasing fiber content up to 1% improves the quality index, while further increases lead to a reduction in performance. Similarly, microsilica content up to 10% enhances the quality index, whereas a decrease is observed at 12.5%. For fiber length, the highest quality index was obtained at 0.5 cm, followed by a decreasing trend.

Accordingly, the combination of 10% microsilica, 1% glass wool fibers, and 0.5 cm fiber length was identified as an optimal region for maximizing the quality index. It should be noted that this optimum region was identified from the experimental observations only. The continuous optimum predicted by the response surface model is presented later in Section 3.2.6.

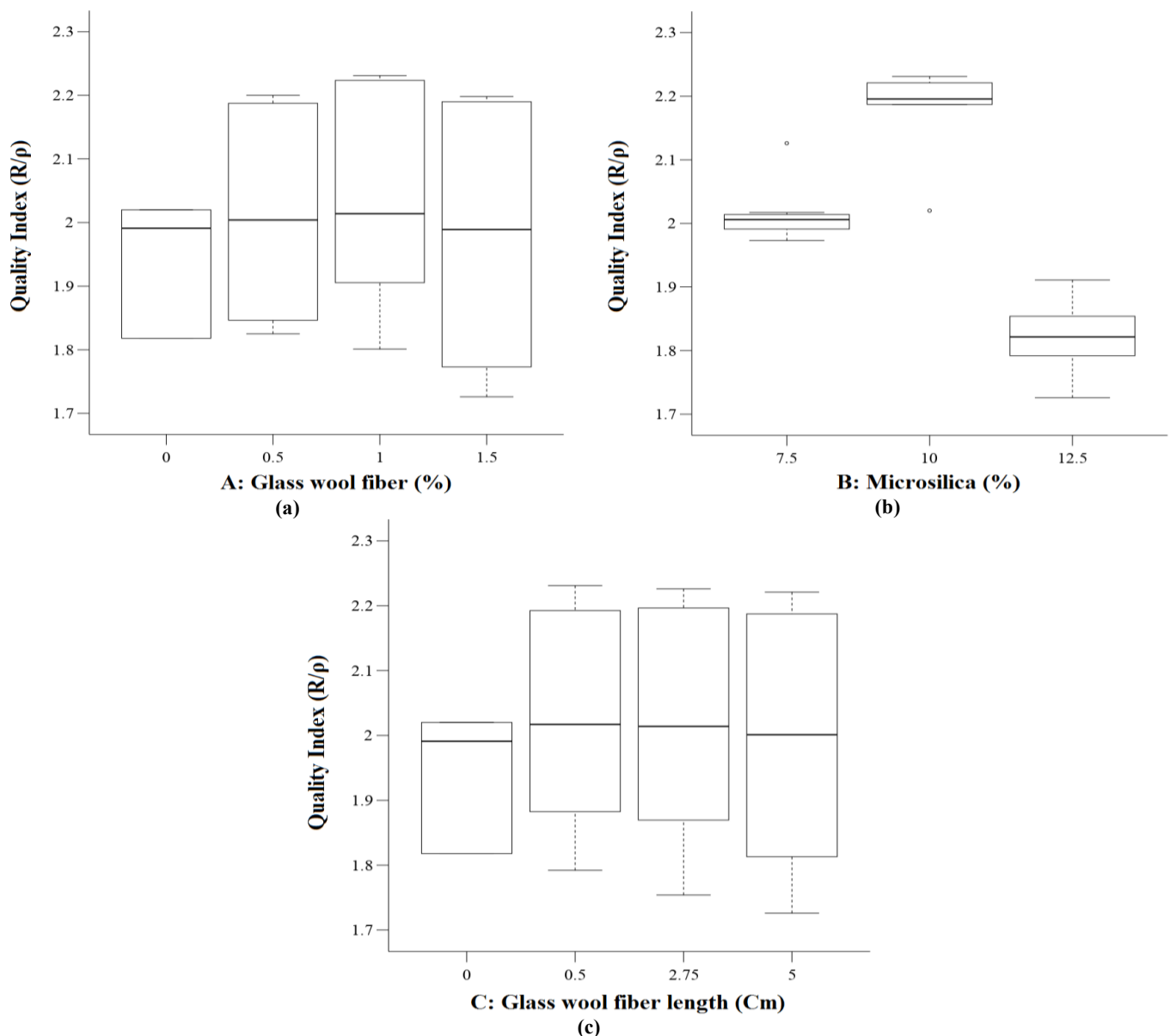


Fig. 8. Boxplots illustrating the distribution of the Quality Index (R/ρ) with respect to the investigated variables: (a) glass wool fiber content; (b) microsilica content; and (c) glass wool fiber length.

3.2.2. Analysis of the quality index data

Several regression models, including linear, two-factor interaction (2FI), quadratic, and cubic models, were evaluated using Design-Expert® (Version 13) [32], and the most appropriate model was selected according to the statistical criteria summarized in

Table 4. The experimental data were analyzed using Response Surface Methodology (RSM) implemented in Design-Expert® (Version 13) [32]. Three independent variables were considered, namely glass wool fiber content (factor A), microsilica content (factor B), and glass wool fiber length (factor C), while the quality index (R/ρ) was selected as the response variable. Linear, two-factor interaction (2FI), quadratic, and cubic polynomial models were evaluated. The final model was selected according to the sequential model sum of squares, model significance, adjusted coefficient of determination (Adjusted R^2), predicted coefficient of determination (Predicted R^2), and model hierarchy.

Table 4. Model selection summary for response surface model selection.

Sequential model sum of squares [type I]					
Source	Sum of squares	df	Mean square	F-value	P-value
Mean vs total	120.83	1	120.83		
Linear vs mean	0.1847	3	0.0616	2.89	0.0547
2FI vs linear	0.0107	3	0.0036	0.1511	0.9279
Quadratic vs 2FI	0.5025	3	0.1675	81.71	< 0.0001
Cubic vs quadratic	0.0269	8	0.0034	2.86	0.0495
Residual	0.0141	12	0.0012		
Total	121.57	30	4.05		
Model summary statistics					
Source	Std. Dev.	R^2	Adjusted R^2	Predicted R^2	PRESS
Linear	0.1460	0.2499	0.1634	0.0309	0.7161
2FI	0.1537	0.2644	0.0725	-0.2477	0.9219
Quadratic	0.0453	0.9445	0.9195	0.8612	0.1026
Cubic	0.0343	0.9809	0.9538	0.8377	0.1199
Fit summary					
Source	Model p-value	Adjusted R^2	Predicted R^2		
Design model	< 0.0001	0.9231	0.8659	Recommended	
Linear	0.0547	0.1634	0.0309		
2FI	0.9279	0.0725	-0.2477		
Quadratic	< 0.0001	0.9195	0.8612	Suggested	
Cubic	0.0495	0.9538	0.8377	Aliased	

Among the evaluated regression models, the quadratic model was selected because it provided the best compromise between prediction accuracy and model simplicity. As shown in Table 4, the quadratic model exhibited a statistically significant p-value (< 0.0001), together with high Adjusted R^2 (0.9195) and Predicted R^2 (0.8612) values, indicating excellent agreement between experimental observations and model predictions. Although the cubic model exhibited a slightly higher coefficient of determination, it was aliased because the available experimental data were insufficient to estimate all higher-order coefficients independently. Therefore, the quadratic model was selected as the most statistically reliable regression model.

After selecting the quadratic model, an analysis of variance (ANOVA) was performed to identify the statistically significant factors affecting the quality index. The significance of each main effect, interaction term, and quadratic term was evaluated at a confidence level of 95% ($p < 0.05$). The ANOVA results of the reduced model are presented in Table 5.

Table 5. Analysis of variance (ANOVA) for the reduced quadratic response surface model.

Source	Sum of squares	df	Mean square	F-value	P-value	Remark
Model	0.6978	8	0.0872	44.53	< 0.0001	Significant
A (glass wool fiber content)	0.0062	1	0.0062	3.18	0.0890	Not significant
B (microsilica content)	0.1606	1	0.1606	82.01	< 0.0001	Significant
C (glass wool fiber length)	0.0066	1	0.0066	3.39	0.0798	Not significant
A × B	0.0006	1	0.0006	0.3152	0.5804	Not significant
A × C	0.0001	1	0.0001	0.0382	0.8469	Not significant
B × C	0.0002	1	0.0002	0.0875	0.7702	Not significant
A ²	0.0262	1	0.0262	13.40	0.0015	Significant
B ²	0.4762	1	0.4762	243.09	< 0.0001	Significant
Residual	0.0411	21	0.0020			
Cor total	0.7389	29				

Note: Factor coding is coded. Sum of squares is type III (partial).

Model statistics			
Statistic	Value	Statistic	Value
Std. Dev.	0.0443	R ²	0.9443
Mean	2.01	Adjusted R ²	0.9231
C.V. %	2.21	Predicted R ²	0.8659
		Adeq precision	19.5336

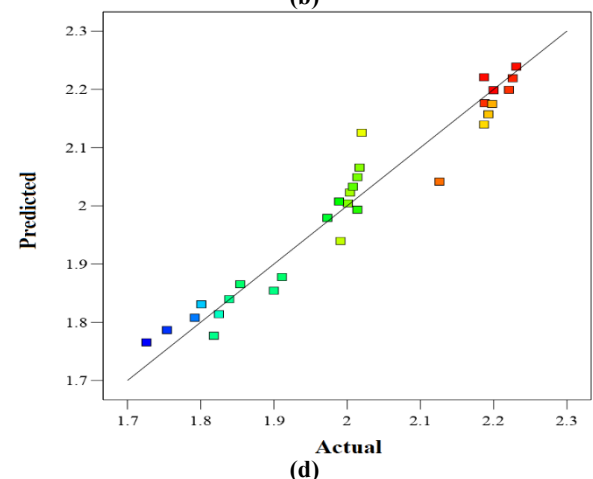
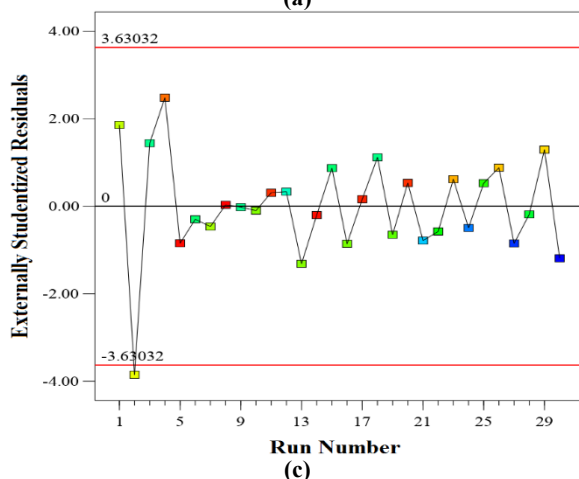
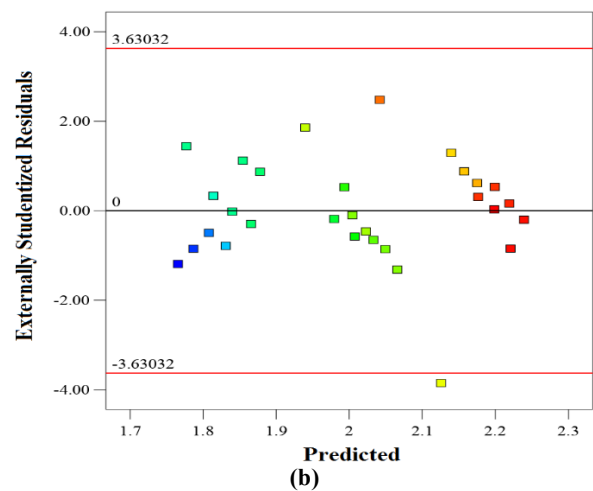
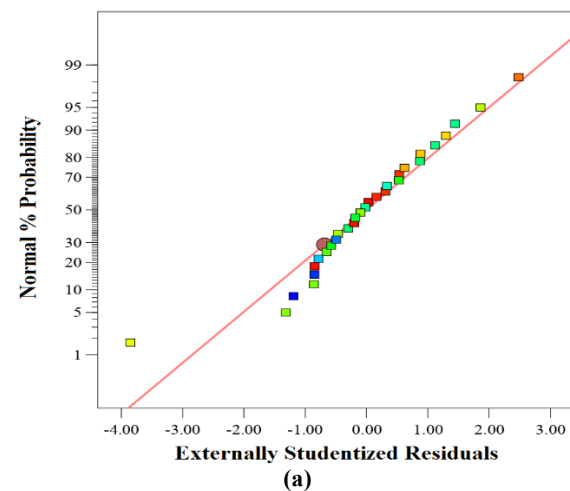
The ANOVA results indicate that microsilica content (factor B) was the most influential variable affecting the quality index ($p < 0.0001$), whereas the linear effects of glass wool fiber content (factor A) and fiber length (factor C) were not statistically significant within the investigated experimental range ($p > 0.05$). Nevertheless, the quadratic effects of fiber content (A^2) and microsilica content (B^2) were statistically significant, confirming the existence of optimum values rather than purely linear relationships. The absence of statistically significant interaction terms (AB, AC, and BC) indicates that the effects of the investigated variables were predominantly independent within the studied design space. Consequently, the overall response was governed mainly by the individual contribution of microsilica content together with the nonlinear behavior of microsilica and glass wool fiber dosage. This finding suggests that the influence of fiber length is more pronounced in post-peak behavior and strain-related responses than in the strength-to-density ratio (R/ρ).

High values of R^2 and Predicted R^2 confirm good model fitting and reliable prediction of new observations. Furthermore, a high Adequate Precision value indicates an adequate signal-to-noise ratio and confirms the suitability of the model for mixture design applications. Overall, the statistical analysis confirms the robustness of the model and supports its application in engineering practice.

The final regression equation expressed in terms of actual factor values is presented below and can be used to predict the quality index within the investigated experimental domain only. Extrapolation beyond the investigated ranges is not recommended.

$$\text{Quality index} = -1.82487 + 0.329672(A) + 0.822649(B) - 0.004593(C) - 0.004701(AB) + 0.002029(AC) - 0.000629(BC) - 0.164751(A^2) - 0.042760(B^2) \quad (3)$$

The regression equation demonstrates that microsilica content has the strongest positive contribution to the quality index, whereas the significant quadratic coefficients indicate the presence of optimum material proportions. Consequently, increasing microsilica or fiber dosage beyond their optimum levels does not continuously improve the response, which agrees well with the experimental observations.



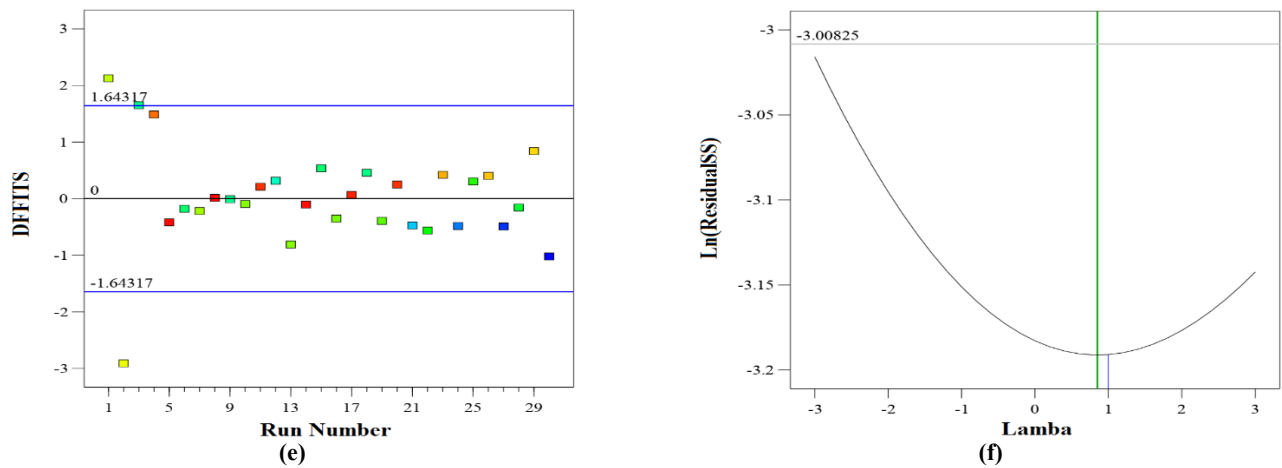


Fig. 9. Diagnostic plots used for validation of the quadratic response surface model: (a) normal probability plot of residuals; (b) residuals versus predicted values; (c) residuals versus run order; (d) predicted versus actual values; (e) DFFITS versus run order; and (f) Box-Cox plot for power transformation.

The diagnostic plots presented in Fig. 9 confirm the adequacy of the selected quadratic model. The normal probability plot indicates that the residuals are approximately normally distributed, while the residual-versus-predicted plot demonstrates a random distribution without systematic trends, confirming the absence of significant heteroscedasticity. Furthermore, the Box-Cox analysis indicates that no transformation of the response variable is required because the confidence interval includes $\lambda = 1$. These diagnostic results demonstrate that the assumptions required for regression analysis were satisfactorily fulfilled and therefore support the reliability of the developed RSM model.

Fig. 10 illustrates the main effects of glass wool fiber content (A), microsilica content (B), and glass wool fiber length (C) on the quality index (R/ρ). The response surface model indicates that microsilica content and glass wool fiber dosage exhibit nonlinear relationships with the quality index, resulting in distinct optimum values of approximately 9.39% and 0.87%, respectively. In contrast, fiber length showed a comparatively smaller influence on the quality index within the investigated range. These trends are consistent with the ANOVA results, which identified the quadratic effects of microsilica content and fiber dosage as statistically significant. It should be emphasized that the optimum values shown in Fig. 10 are mathematical predictions obtained from the continuous RSM model and should not be interpreted as experimentally tested mixtures. They represent interpolated optimum conditions within the investigated design space.

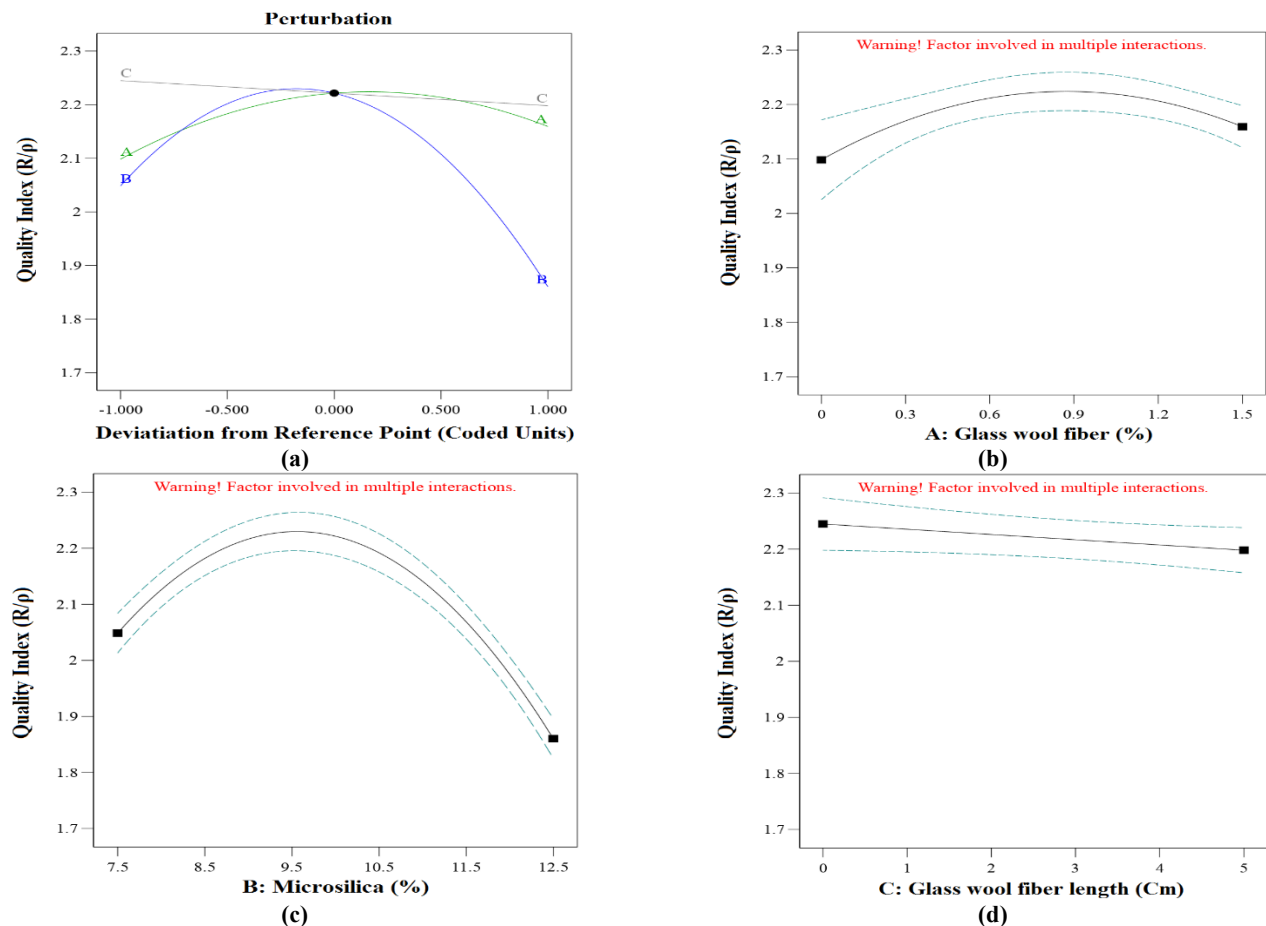


Fig. 10. Main effect plots showing the influence of the investigated variables on the quality index (R/ρ): (a) combined effects of all factors; (b) glass wool fiber content; (c) microsilica content; and (d) glass wool fiber length.

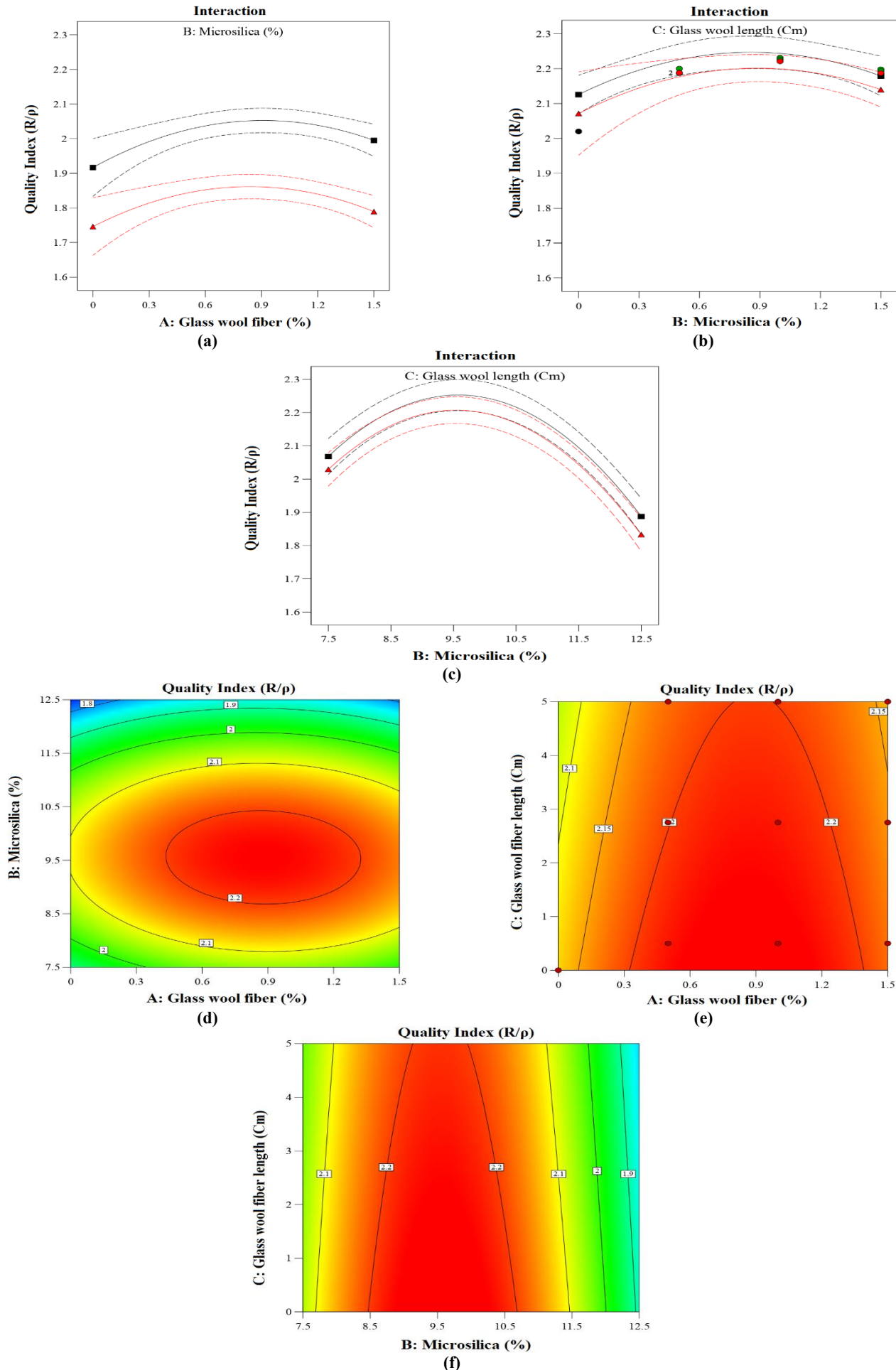


Fig. 11. Interaction and contour plots illustrating the effects of the investigated variables on the Quality Index (R/ρ): (a) interaction plot of factors A and B; (b) interaction plot of factors A and C; (c) interaction plot of factors B and C; (d) contour plot of factors A and B; (e) contour plot of factors A and C; and (f) contour plot of factors B and C.

Fig. 11 presents the contour plots describing the interaction between the investigated variables and the predicted quality index. Although the interaction terms were not statistically significant according to the ANOVA results, the contour plots provide a useful graphical representation of the response surface and facilitate visualization of the optimum design region. The red circles indicate the experimentally tested mixtures, whereas the continuous contours represent the response predicted by the RSM model.

Fig. 12 illustrates the three-dimensional response surfaces generated by the quadratic regression model. The surfaces predict the variation of the quality index throughout the investigated experimental domain. Therefore, the predicted optimum points should be interpreted as model estimations rather than direct experimental observations.

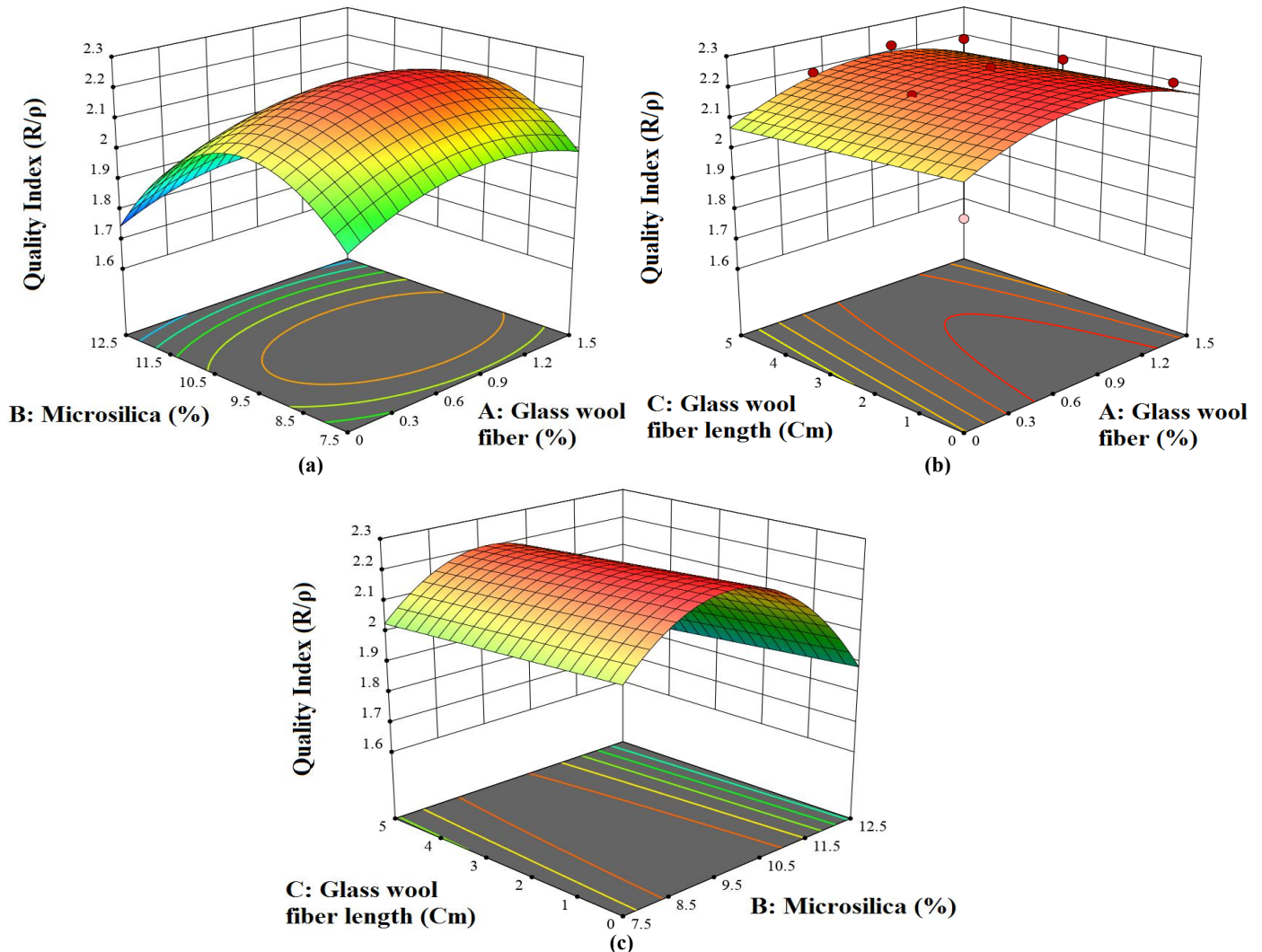
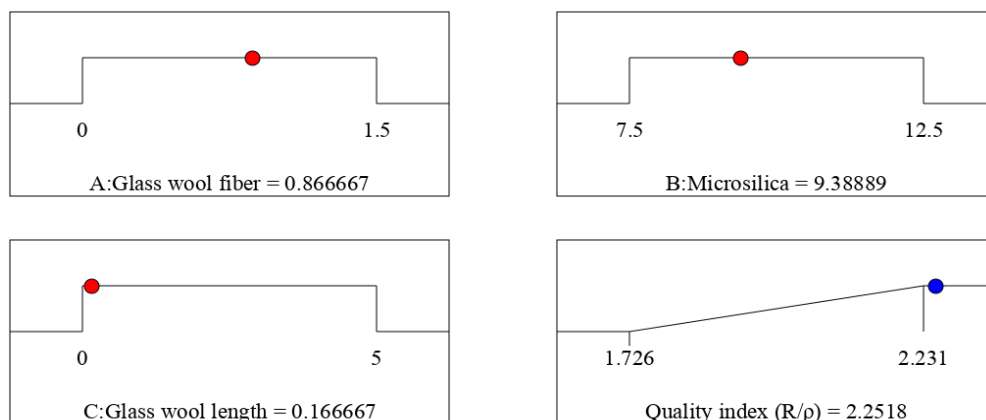


Fig. 12. (a) Three-dimensional response surface of glass wool fiber content and microsilica content; (b) glass wool fiber content and fiber length; and (c) microsilica content and glass wool fiber length.

The optimization results obtained from the RSM model are presented in Fig. 13. Two different optimum conditions should be clearly distinguished. The highest experimentally measured compressive strength (40.0 MPa) was obtained for the mixture containing 10% microsilica, 0.5% glass wool fibers, and a fiber length of 27.5 mm.



Desirability = 1.000
Solution 17 out of 100
(a)

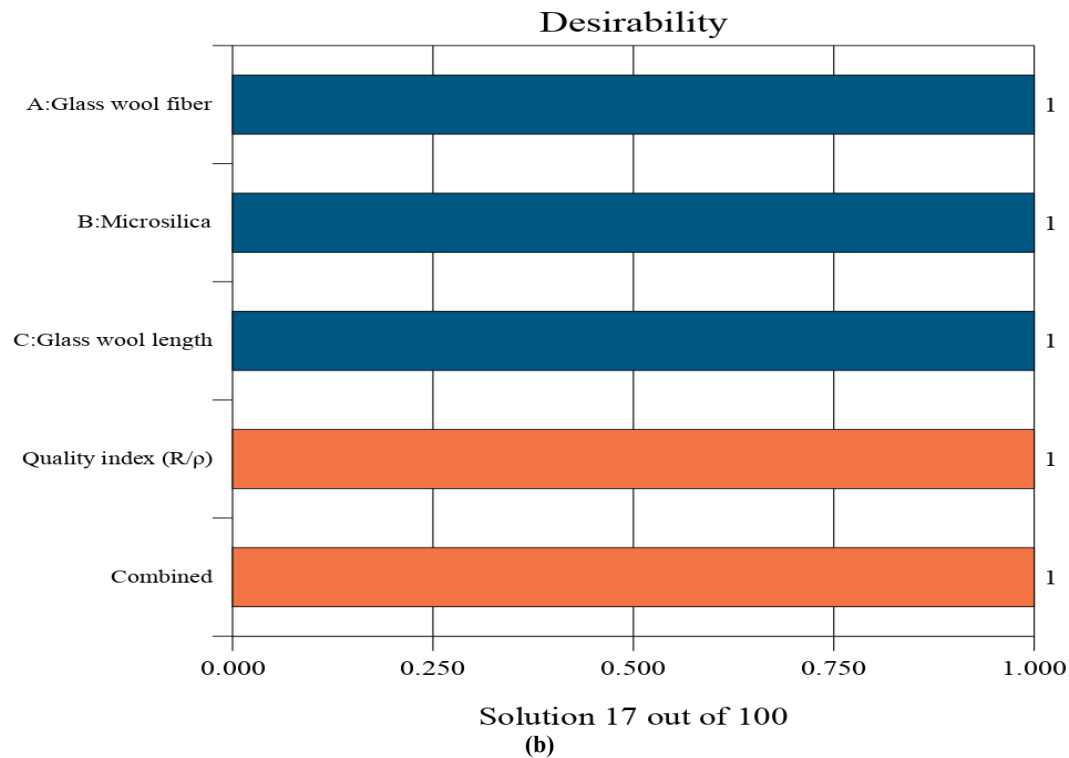


Fig. 13. Results of the RSM optimization: (a) numerical optimization showing the optimum levels of glass wool fiber content, fiber length, and microsilica content for maximizing the Quality Index (R/ρ); and (b) desirability function corresponding to the optimum solution.

However, the multi-objective optimization performed using the RSM model predicted a different optimum mixture containing approximately 9.39% microsilica and 0.87% glass wool fibers, which maximized the quality index ($R/\rho \approx 2.25$). This difference is expected because the experimental optimum corresponds to maximizing compressive strength alone, whereas the RSM optimization simultaneously considers both compressive strength and concrete density. Consequently, the optimum quality index represents the highest strength-to-density efficiency rather than the highest compressive strength. Accordingly, the experimentally determined optimum mixture and the RSM-optimized mixture represent two different optimization objectives derived from the same experimental database.

Fig. 14 presents the influence of the investigated variables on the desirability function used during numerical optimization. The desirability value approaches unity within the optimum design region, indicating that the selected combination of variables simultaneously satisfies the optimization criteria defined for the response variable.

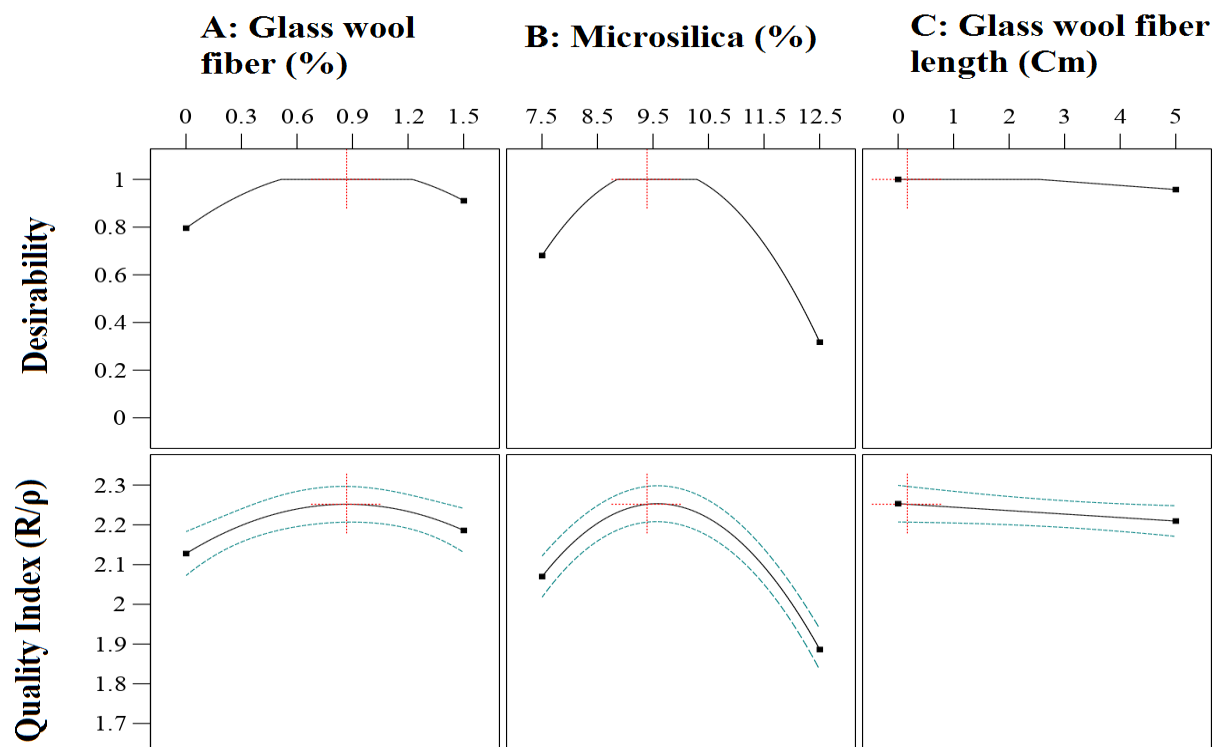


Fig. 14. Main effect plots showing the influence of glass wool fiber content (%), microsilica content (%), and glass wool fiber length (Cm) on the desirability function.

The contour plots in Fig. 15 illustrate the interaction effects among the variables in the design space. To evaluate the predictive capability of the developed RSM model, three additional concrete mixtures outside the model calibration dataset were experimentally tested. The differences between the experimentally measured and RSM-predicted quality index values were less than 4%, confirming the satisfactory predictive capability and robustness of the developed regression model within the investigated design space. Nevertheless, additional validation using independent experimental datasets covering a wider range of mixture proportions is recommended to further improve the general applicability of the proposed predictive model.

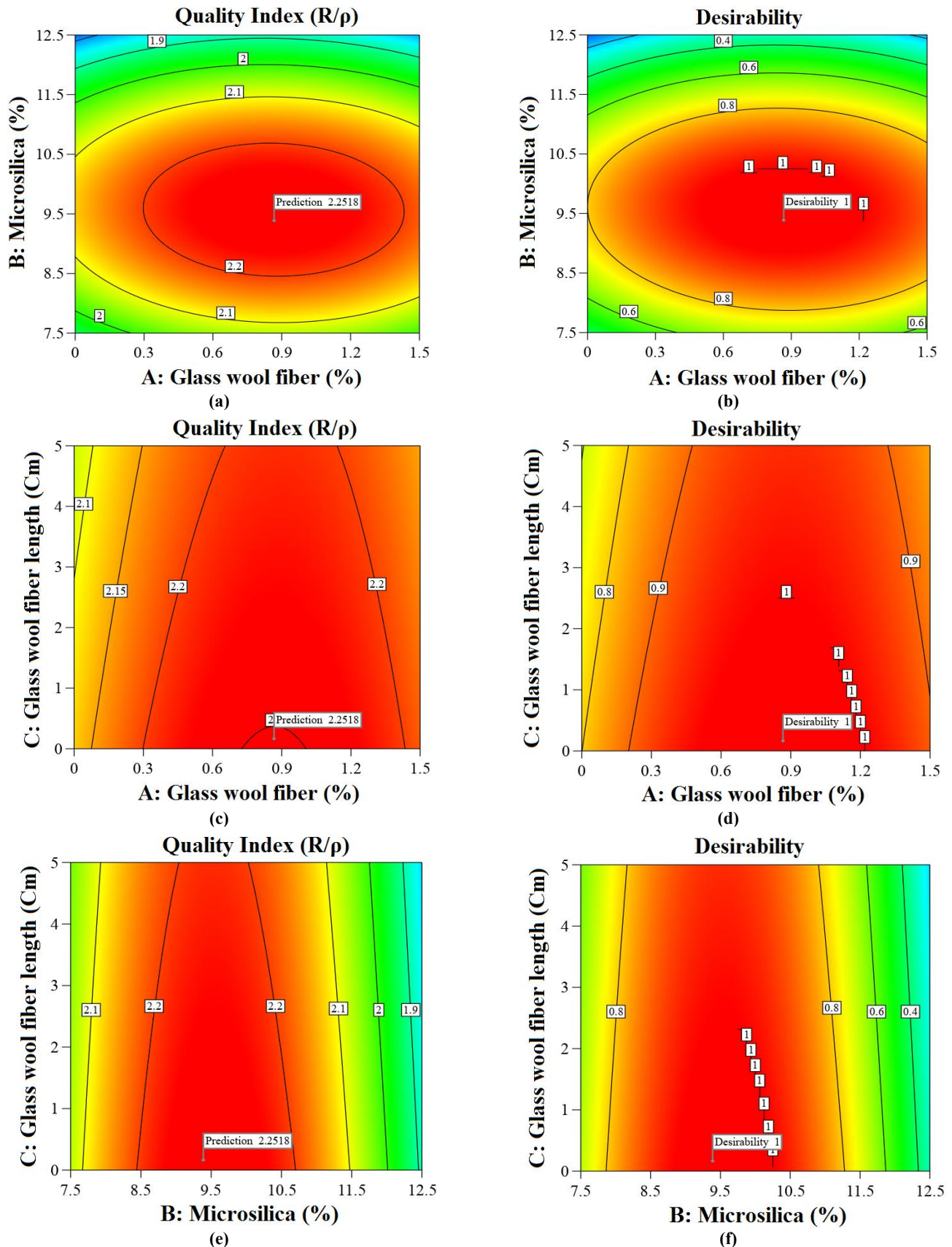


Fig. 15. Contour plots of the response and desirability functions for the investigated factor interactions: (a) quality index (R/ρ) for factors A and B; (b) desirability for factors A and B; (c) quality index (R/ρ) for factors A and C; (d) desirability for factors A and C; (e) quality index (R/ρ) for factors B and C; and (f) desirability for factors B and C.

Previous studies have reported that the incorporation of approximately 5% microsilica can enhance compressive strength by nearly 12% in fiber-reinforced concrete [53]. In the present study, the combination of 10% microsilica and 0.5% Glass wool fibers resulted in a 7.5% increase in compressive strength of lightweight concrete. The difference is mainly attributed to the higher porosity of perlite aggregates and reduced matrix continuity in lightweight concrete. Additionally, the lower water-to-cement ratio (0.335) and reduced density limit the absolute load-bearing capacity. Nevertheless, the results confirm the beneficial role of microsilica in densifying the matrix and improving the interfacial transition zone (ITZ), which is consistent with the findings of Singaravel et al. (2024) [52].

The fiber content analysis revealed an optimum value of approximately 0.5%, beyond which mechanical performance decreased due to fiber agglomeration and reduced workability. This behavior is consistent with previous studies on fiber-reinforced concrete systems [40]. The increase in fiber content beyond 1% led to strength reduction and higher porosity due to non-uniform dispersion and matrix discontinuity. Therefore, simultaneous optimization of microsilica and fiber content is essential not only for compressive strength but also for the mechanical efficiency index (R/ρ) [45]. Overall, the results demonstrate that optimizing microsilica and glass wool fiber contents effectively improves the mechanical efficiency of lightweight concrete.

4. Conclusions

Based on the experimental investigation and response surface methodology (RSM) analysis, the following conclusions can be drawn:

- Thirty perlite lightweight concrete mixtures containing different microsilica contents (7.5-12.5%), glass wool fiber dosages (0-1.5 vol%), and fiber lengths (5-50 mm) were experimentally evaluated to investigate their effects on compressive strength, stress-strain behavior, energy absorption, and quality index (R/ρ).
- The highest compressive strength (40.0 MPa) was obtained for the mixture containing 10% microsilica and 0.5% glass wool fibers with a fiber length of 27.5 mm, corresponding to an approximately 7.5% increase compared with the control mixture.
- Increasing the glass wool fiber dosage improved ductility, ultimate strain, and post-peak energy absorption. The greatest deformation capacity was observed at 1.5% fiber dosage; however, excessive fiber addition reduced workability and matrix homogeneity, leading to a slight reduction in compressive strength.
- The incorporation of approximately 10% microsilica significantly improved the mechanical performance through matrix densification, pore refinement, and enhancement of the interfacial transition zone (ITZ). Higher replacement levels (12.5%) resulted in a reduction in mechanical performance, mainly due to increased water demand and reduced stress redistribution capacity.
- ANOVA results confirmed that microsilica content (factor B) together with the quadratic effects of fiber dosage (A^2) and microsilica content (B^2) significantly influenced the quality index ($p < 0.05$), whereas the linear effects of fiber length and the interaction terms were statistically insignificant within the investigated design space.
- The experimentally determined optimum mixture for maximum compressive strength differed from the RSM-predicted optimum for the quality index. While the RSM optimization predicted a mixture containing approximately 9.39% microsilica and 0.87% glass wool fibers that yielded the highest quality index ($R/\rho \approx 2.25$), the experimental mixture produced the maximum compressive strength. This distinction demonstrates that maximizing compressive strength and maximizing strength-to-density efficiency are two different optimization objectives.
- SEM observations confirmed that the combined incorporation of microsilica and glass wool fibers improved matrix compactness, strengthened the interfacial transition zone, and promoted effective crack-bridging mechanisms, which explain the observed improvements in ductility, post-peak behavior, and energy absorption capacity.
- The novelty of this study lies in integrating experimental investigation with RSM-based multi-objective optimization to distinguish between experimentally measured optimum mechanical performance and mathematically predicted optimum strength-to-density efficiency. The proposed approach provides a practical framework for optimizing structural lightweight perlite concrete within the investigated material ranges. These conclusions are applicable only to the material ranges and experimental conditions considered in this study.

5. Limitations and recommendations

- This study was conducted under laboratory conditions and primarily focused on compressive strength and quality index. Therefore, long-term durability, environmental exposure, and economic considerations were not investigated.
- In the RSM analysis, glass wool fiber dosage and fiber length were treated as continuous variables, whereas the experimental program was conducted only at discrete factor levels. Consequently, the predicted optimum values (including the 0.87% fiber dosage and intermediate fiber lengths) represent mathematical interpolations within the investigated design space rather than experimentally verified optimum mixtures. Therefore, additional experiments are recommended to validate these predicted optimum conditions.
- Only one type of lightweight aggregate (perlite) and one type of glass wool fiber were employed in this study. Future

research is recommended to investigate other lightweight aggregates and fiber types, as well as the effects of environmental conditions such as freeze-thaw cycles, sulfate and alkaline exposure, and alternative pozzolanic materials on the performance of lightweight concrete.

Statements & declarations

Author contributions

Hooman Yaghebipour: Conceptualization, Methodology, Investigation, Formal analysis, Data curation, Visualization, Writing - Original Draft, Writing - Review & Editing.

Hamidreza Tavakoli: Conceptualization, Methodology, Validation, Resources, Supervision, Project administration, Writing - Review & Editing.

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Data availability

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

Declarations

The authors declare no conflict of interest.

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Compressive Strength and Sustainability Assessment of Mortar Incorporating Rice Husk Biochar as a Sustainable Cement Replacement

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ABSTRACT

The cement industry is a major contributor to global CO₂ emissions, highlighting the need for sustainable supplementary cementitious materials (SCMs) capable of reducing cement consumption while maintaining mechanical performance. This study investigated the effects of raw rice husk (SH), rice husk biochar (BSH), and calcined rice husk ash (CSH) as partial replacements for ordinary Portland cement. Mortar mixtures incorporating 3%, 5%, and 8% of each material were evaluated in terms of workability, compressive strength, normalized compressive strength, and strength development up to 28 days. In addition, a cradle-to-gate life cycle assessment (LCA) was performed to quantify greenhouse gas (GHG) emissions, cumulative energy demand (CED), and environmental impacts using the ReCiPe endpoint method. The results showed that the processing method of rice husk significantly influenced both mechanical and environmental performance. Among the investigated materials, CSH improved workability because of its filler effect and fine particle size. The mixture containing 3% CSH and BSH achieved the highest 28-day compressive strength (43.4 and 41.1 MPa, respectively), corresponding to 143.6% and 136.0% of the reference mixture's compressive strength (i.e., a 43.6% and 36.0% increase), respectively. Biochar enhanced long-term strength through internal curing, whereas SH exhibited slower early-age strength development but substantial later-age strength gain. The LCA results indicated that replacing 8% of cement with RH reduced CO₂ emissions, cumulative energy demand, and the overall ReCiPe endpoint impact by 7.7%, 6.7%, and 4.8%, respectively, compared with the control mixture, while CSH also provided considerable environmental benefits. Overall, SH-derived materials offer an effective strategy for producing low-carbon cementitious mortars. The 3% CSH mixture delivered the best mechanical performance, whereas the 8% SH mixture achieved the greatest environmental benefits, demonstrating the importance of balancing engineering performance and sustainability.

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1. Introduction

Rice is one of the world's major agricultural products, with an annual production exceeding 750 million tons. Alongside this production, approximately 150 million tons of rice husk (SH) are generated as a by-product of rice processing [1-3]. In many rice-producing regions, including northern Iran, this residue is still predominantly managed through open-field burning or accumulation on agricultural lands. Such practices not only contribute to air pollution and particulate emissions but also remove a significant

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economic opportunity from the production cycle [4]. In recent years, the concept of the bio-circular economy has gained increasing attention as a framework for recovering value from agricultural residues. Consequently, SH is no longer regarded as a worthless waste material but rather as a valuable feedstock capable of being transformed into a wide range of products for the energy, agricultural, and construction sectors [5, 6].

Among the industries utilizing agricultural residues, the construction sector represents one of the primary destinations for such by-products due to its extensive consumption of raw materials and substantial contribution to greenhouse gas emissions. Portland cement, the most widely used binder in mortar and concrete, remains the most consumed manufactured material worldwide because of its compatibility, low maintenance requirements, and abundant raw material availability. However, it is also responsible for approximately 7-8% of global anthropogenic carbon dioxide emissions [7, 8]. It has been reported that the production of one ton of Portland cement clinker releases approximately 0.8 tons of CO₂ into the atmosphere [9]. This environmental concern has driven researchers toward the development of supplementary cementitious materials (SCMs), which can partially replace cement while maintaining or even enhancing the mechanical performance and durability of the final product [10-12]. Owing to their abundance, renewability, and relatively low production cost compared with other cement replacement materials such as metakaolin or blast furnace slag, agricultural waste-derived products have emerged as attractive SCM candidates [13].

SH, in particular, has long been recognized as a promising source for the production of supplementary cementitious materials due to its naturally high silica content. However, the conversion of this raw agricultural residue into a cementitious material can be achieved through different thermochemical pathways, each yielding products with distinct physicochemical characteristics and performance mechanisms. The most common route is calcination (controlled combustion in the presence of oxygen), which produces calcined rice husk ash (CSH). The pozzolanic quality of CSH is highly dependent on the calcination temperature, as processing parameters such as temperature, duration, and atmospheric conditions directly influence the amount of amorphous silica, specific surface area, and particle morphology of the resulting ash [14]. Previous studies have demonstrated that combustion within a temperature range of approximately 500-700°C preserves silica predominantly in its amorphous form and results in the highest pozzolanic activity. In contrast, higher temperatures promote the gradual crystallization of silica (forming cristobalite), leading to a reduction in pozzolanic reactivity [15-17].

Conversely, pyrolysis conducted under oxygen-limited or oxygen-free conditions produces rice husk biochar (BSH) instead of ash. BSH is a carbon-rich, porous, and stable material that, in addition to its well-established applications in soil amendment and carbon sequestration, has recently attracted attention for use in cementitious composites [18, 19]. Unlike CSH, whose contribution is primarily pozzolanic, the role of biochar within the cement matrix is largely physicochemical. Its influence is exerted through mechanisms such as particle-filling effects, internal curing resulting from water absorption and gradual release, and serving as nucleation sites for hydration products, thereby affecting both the microstructure and mechanical properties of mortar [20, 21].

Numerous studies have independently investigated the mechanical performance of these two rice husk-derived products; however, the reported findings remain relatively scattered and, in some cases, contradictory due to differences in processing conditions, particle size, and replacement levels. For CSH, several studies have reported compressive strength increases exceeding 50% at replacement levels of 15-25%, attributed to the formation of additional C-S-H and C-A-S-H gels, microstructural densification, and improvement of the interfacial transition zone (ITZ) [22]. Similarly, for rice husk biochar, a range of results has been documented in the literature, varying from approximately 12-14% improvements in compressive strength at replacement levels of 5-10%, attributed to filler effects and reduced pore connectivity [18], to strength reductions in mixtures characterized by high porosity or low contents of reactive silicon and aluminum oxides [23]. This variability indicates that the selected thermochemical processing route for rice husk plays a decisive role in determining its effectiveness as a cement replacement material. In addition to its mechanical and environmental performance under ambient conditions, rice husk ash-modified concrete has also been shown to exhibit favorable behavior under thermal exposure, with water re-curing partially recovering strength losses following exposure to elevated temperatures of 300-700°C [24].

Although numerous investigations have focused on the mechanical and durability performance of rice husk-derived supplementary cementitious materials, structural performance alone is insufficient to determine their overall sustainability. For instance, recent studies have shown that rice husk ash-modified concrete exhibits favorable resistance to thermal degradation, with water re-curing partially recovering strength losses following exposure to elevated temperatures [24], and a complementary life cycle assessment of the same material further confirming its favorable environmental profile under such exposure conditions [25]. The thermochemical conversion routes required to produce rice husk ash and rice husk biochar differ considerably in terms of energy demand, process emissions, and resource consumption, which may significantly influence their environmental benefits despite comparable engineering performance. In recent years, life cycle assessment (LCA) has become one of the most widely accepted methodologies for quantifying the environmental impacts of construction materials throughout their life cycle, enabling comprehensive comparisons of alternative materials based on indicators such as global warming potential, cumulative energy demand, and damage to human health, ecosystems, and natural resources. Consequently, integrating mechanical performance with life cycle assessment provides a more reliable basis for identifying sustainable cement replacement strategies and supports decision-making toward low-carbon construction materials [26-29].

Despite the growing body of literature addressing SH or BSH individually [17, 19, 21], no study has yet simultaneously compared all three thermochemical processing routes of SH (SH, CSH, and BSH) within a unified experimental and environmental framework. The novelty of the present study lies in this three-way comparison combined with an integrated mechanical-environmental assessment: by evaluating compressive strength development at multiple curing ages alongside a cradle-to-gate LCA for each

processing route, this study identifies not only which material performs best mechanically, but which replacement level and processing route offers the most balanced trade-off between structural performance and environmental sustainability. This integrated approach, rather than either aspect alone, provides a more practical basis for material selection in low-carbon cementitious design. Therefore, this study aims to comparatively investigate the compressive strength development of mortar incorporating CSH and BSH as partial cement replacements at different curing ages. In addition, a cradle-to-gate life cycle assessment is conducted to quantify the environmental impacts associated with each processing pathway and to identify the more sustainable alternative considering both mechanical performance and environmental criteria.

2. Methodology

2.1. Materials

In the present study, Type II Portland cement produced by Mazandaran Cement Company was used as the primary binder. A recycled rice husk-derived material was introduced as a SCM in three different forms, namely SH, BSH, and CSH, for the production of low-carbon cement (Fig. 1). As shown in Fig. 1, the three forms of rice husk-derived material exhibit visibly distinct physical characteristics: raw rice husk retains its fibrous, light-tan appearance; the biochar exhibits a dark black, porous texture resulting from carbonization under oxygen-limited conditions; and the calcined ash presents a fine, light-gray powder consistent with the removal of carbonaceous material during high-temperature oxidation. The chemical compositions of the Portland cement, raw rice husk, rice husk biochar, and calcined rice husk ash used in this study are presented in Table 1. Chemical compositions were determined by X-ray fluorescence (XRF) spectroscopy, and loss-on-ignition (LOI) values were obtained by heating samples at 1100°C for 2 hours and measuring the resulting mass loss. The SH employed in this research were obtained from Tarom rice cultivated in the Sarbaz region of Gilan Province, representing one of the common rice varieties available in northern Iran. As shown in Table 1, the SiO₂ content increased progressively from SH (78.8%) to BSH (87.2%) and CSH (91.0%), reflecting the progressive removal of organic matter and volatile compounds during thermal treatment. The relatively high K₂O content observed in BSH (4.7%) compared with the other materials should be noted, as elevated alkali content can potentially influence the risk of alkali-silica reaction (ASR) when used at higher replacement levels. The low CaO content across all three SH-derived materials (0.6–1.8%), in contrast to Portland cement (63.2%), confirms their primarily pozzolanic (rather than hydraulic) nature, consistent with their role as reactive silica sources rather than independent binders. The CSH was subjected to thermal treatment at a temperature range of 500–550°C for one hour. This calcination temperature was selected to prevent incomplete combustion, eliminate residual organic matter, and obtain an amorphous silica-rich structure, since the presence of amorphous silica is the primary factor governing effective pozzolanic reactions at later curing ages [30]. Calcination temperatures below the optimum range may result in the retention of unburned carbon and consequently reduce pozzolanic activity, whereas excessively high temperatures can promote silica crystallization and diminish its reactivity. After calcination at 500–550°C for one hour, the resulting ash was ground and sieved to pass through a 75 µm (No. 200) sieve, consistent with the particle fineness applied to the biochar and comparable to that of Portland cement, to ensure comparable reactivity and particle-filling behavior among the three processing routes.

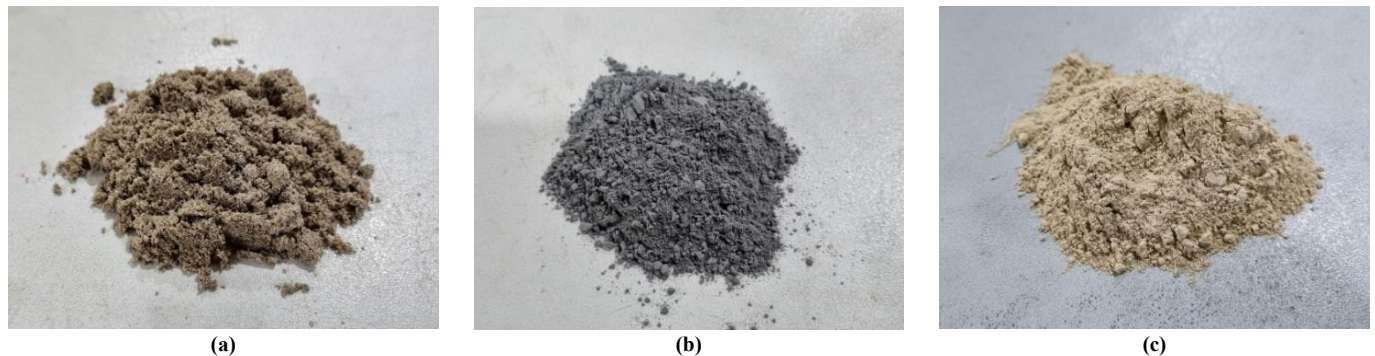


Fig. 1. Physical appearance of the three RH-derived materials: (a) SH; (b) BSH; and (c) CSH.

Table 1. Cementing materials' chemical compositions.

	SiO ₂	CaO	Al ₂ O ₃	Fe ₂ O ₃	MgO	Na ₂ O	K ₂ O	SO ₃	Ti ₂ O	LOI
Cement	21.3	63.2	4.9	3.4	1.2	0.3	0.6	2.3	-	2.5
Raw rice husk (SH)	78.8	0.6	0.2	0.5	0.3	0.05	2.1	0.1	0.02	16.9
Rice husk biochar (BSH)	87.2	1.8	0.9	1.4	0.5	-	4.7	0.15	0.23	3.03
Calcined rice husk (CSH)	91.0	0.8	1.1	1.3	0.2	-	0.9	-	-	-

BSH was produced through a controlled pyrolysis process in a muffle furnace under oxygen-limited conditions at approximately 450–500°C for about 1–2 hours, with a heating rate of approximately 10°C/min. After pyrolysis, the biochar was ground and sieved to pass through a 75 µm (No. 200) sieve to achieve a fineness comparable to that of Portland cement. Owing to its porous structure, high specific surface area, and the presence of surface functional groups, this material can influence not only the filler effect but also the microstructure of the cement paste and the paste-aggregate ITZ. These SH-derived materials were utilized as partial cement replacements for the development of low-carbon cement and concrete. Their primary functions within the cementitious system include particle packing and filler effects, nucleation of hydration products, and, in the case of CSH, pozzolanic reactions with the calcium hydroxide released during the cement hydration process.

2.2. Mortar mix designs

The details of the reference sand-cement mortar mixture and the mortars incorporating SH in its raw, biochar, and calcined forms are presented in Table 2. In these mixtures, SH-derived materials were used as partial cement replacements at replacement levels of 3%, 5%, and 8% by weight of the cementitious binder. These replacement levels were selected to maintain adequate workability while enabling a systematic evaluation of the progressive effects of cement substitution on the mechanical performance and microstructural characteristics of the mortar. The selected dosage range also allowed the investigation of both the beneficial and potential adverse effects associated with increasing levels of SH-derived materials within the cementitious matrix. In all mix designs, the water-to-cementitious materials ratio was kept constant at 0.42, and the sand-to-cementitious materials ratio was fixed at 2.0. Maintaining these proportions ensures that the observed variations in mortar properties are primarily attributed to the presence and type of SH-derived material rather than changes in the fundamental mix design parameters. This approach enables a more accurate evaluation of the roles of filler effect, pozzolanic reaction, and microstructural changes induced by the SH. Keeping the sand content constant also helps control the influence of rheological variations in the cement paste and maintain consistency in experimental conditions. This is particularly important because fine rice husk particles, especially in calcined and biochar forms, can increase specific surface area and consequently affect the water demand of the mixture. The particle size distribution of the sand used in this study was determined in accordance with ASTM C136 [31] (sieve analysis), and the particle size distribution curve is presented in Fig. 2. The curve indicates a well-graded distribution of particles, which can lead to improved particle packing and a reduction in capillary porosity in mortar.

Table 2. Mix design of mortar samples.

Mix ID.	Cement	SH	BSH	CSH	Water	Sand	Density
	(g)						(kg/m ³)
C ₁₀₀	1000	-	-	-	420	2000	2481
C ₉₇ SH ₃	970	30	-	-	420	2000	2093
C ₉₅ SH ₅	950	50	-	-	420	2000	2180
C ₉₂ SH ₈	920	80	-	-	420	2000	2254
C ₉₇ BSH ₃	970	-	30	-	420	2000	2300
C ₉₅ BSH ₅	950	-	50	-	420	2000	2288
C ₉₂ BSH ₈	920	-	80	-	420	2000	2336
C ₉₇ CSH ₃	970	-	-	30	420	2000	2315
C ₉₅ CSH ₅	950	-	-	50	420	2000	2260
C ₉₂ CSH ₈	920	-	-	80	420	2000	2285

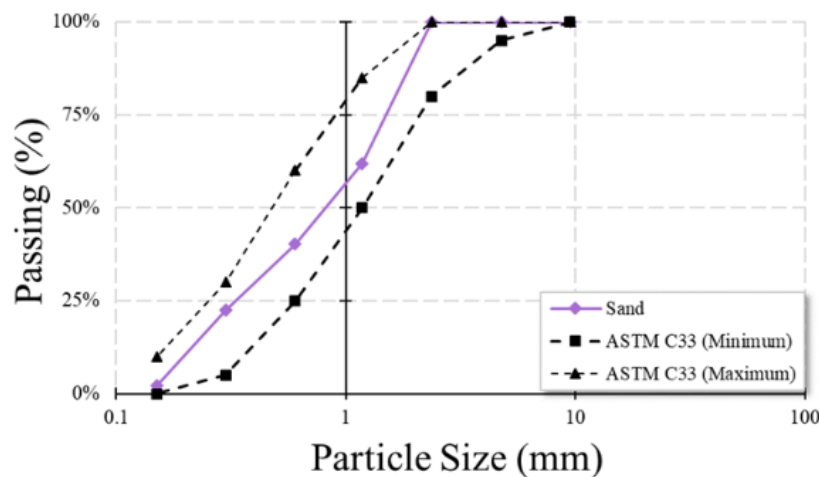


Fig. 2. Particle size distribution.

2.3. Preparation, casting, and curing of mortar specimens

Mortar mixing was performed in accordance with ASTM C305 [32]. To prepare the sand-cement mortar specimens according to Table 2, all materials were mixed using a 10-liter laboratory mixer available in the Concrete Technology Laboratory at Babol Noshirvani University of Technology. First, all cementitious materials, including cement and SH-derived material (raw, biochar, or calcined), were dry-mixed to achieve a homogeneous blend. Subsequently, the calculated amount of water was added to the mixer to form the cement paste. Then, the sand was introduced into the cement paste, and mixing was continued for one minute. When necessary, a superplasticizer was added within 20 to 30 seconds of mixing to achieve the desired workability. A polycarboxylate ether (PCE)-based superplasticizer was used, at dosages adjusted per mix (as % of binder mass) to achieve comparable workability. Superplasticizer was required only for SH mixtures (0.5%, 1.5%, and 2.25% at 3%, 5%, and 8% replacement, respectively), reflecting the higher water demand of untreated, fibrous SH. No superplasticizer was needed for BSH or CSH mixtures, indicating that thermal processing reduced water/admixture demand relative to SH. The workability of the fresh mortar mixtures was evaluated

using the Flow Table test in accordance with ASTM C230 [33]. According to ASTM C511 [34], curing of mortar specimens is a process during which moisture loss is prevented, and an appropriate temperature is maintained. In this study, 24 hours after casting, the specimens were demolded and immediately placed in a water tank maintained at laboratory ambient temperature to preserve surface moisture. The samples were then cured in this tank for 7, 14, and 28 days (Fig. 3). For each mix design, two cubic specimens with dimensions of $50 \times 50 \times 50$ mm were cast and tested at each curing age (a total of six specimens per mix design), and the average value is reported. Compressive strength tests were carried out in accordance with ASTM C109 [35], using an ELE compression testing machine with a capacity of 200 kN. Prior to testing, the bearing surfaces of each specimen were wiped clean to remove any loose material or surface irregularities, ensuring uniform load distribution during testing. Fig. 3a illustrates the casting of fresh mortar into oiled steel molds, and Fig. 3b illustrates the subsequent water curing of demolded specimens, carried out in accordance with the curing protocol described above.



Fig. 3. Casting and curing of mortar specimens.

A complete summary of the experimental specimens, including mix designations, replacement levels, specimen dimensions, and the number of specimens tested at each curing age, is provided in Table 3. In total, 60 cubic specimens ($50 \times 50 \times 50$ mm) were cast and tested for compressive strength across the ten mix designs and three curing ages, with two specimens tested per age and the reported values representing their average. In addition, one flow spread measurement was conducted for each mix design to assess fresh-state workability.

Table 3. Specimen details.

Mix ID	Cement (%)	SH (%)	BSH (%)	CSH (%)	Specimen type/dimensions	Test	Ages/repetitions
C ₁₀₀	100	–	–	–	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₇ SH ₃	97	3	–	–	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₅ SH ₅	95	5	–	–	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₂ SH ₈	92	8	–	–	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₇ BSH ₃	97	–	3	–	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₅ BSH ₅	95	–	5	–	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₂ BSH ₈	92	–	8	–	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₇ CSH ₃	97	–	–	3	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₅ CSH ₅	95	–	–	5	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)
C ₉₂ CSH ₈	92	–	–	8	Cube, 50×50×50 mm	Compressive strength	7, 14, 28 d, 2 specimens/age (6 total)

2.4. Life cycle assessment (LCA)

To evaluate the environmental impacts of replacing cement with three SH-derived materials, namely SH, BSH, and CSH, an LCA was conducted in accordance with the framework specified in ISO 14040 [26] and ISO 14044 [36] standards. The primary objective of this analysis was to compare the greenhouse gas emissions associated with the production of the different mortar mixtures and to identify the most environmentally sustainable cement replacement while considering the corresponding mechanical performance. Since the cement industry is recognized as one of the largest contributors to global carbon dioxide emissions, reducing Portland cement consumption through the incorporation of agricultural by-products has considerable potential to mitigate the environmental impacts of cementitious materials.

In this study, a cradle-to-gate system boundary was adopted, encompassing raw material extraction, transportation, and mortar production processes [37]. The use phase and end-of-life stage were excluded from the assessment, consistent with the scope commonly adopted in material-oriented mortar LCA studies. The functional unit was defined as 1 m³ of mortar, allowing environmental impacts to be expressed in terms of kg CO₂-eq/m³ and other environmental indicators [38]. The environmental assessment was carried out using SimaPro software, while the life cycle inventory (LCI) data were primarily obtained from the Ecoinvent database using global and regional datasets (GLO/RoW) [39]. Transportation impacts were estimated using the "market for" datasets available in Ecoinvent, which incorporate representative transportation distances and average vehicle emission factors based on global conditions.

The life cycle impact assessment (LCIA) was performed using the Greenhouse Gas (GHG) Protocol to quantify carbon dioxide emissions and the 100-year Global Warming Potential (GWP100). In addition, the ReCiPe impact assessment method was employed to evaluate aggregated endpoint indicators related to human health, ecosystem quality, and resource depletion [40, 41]. Furthermore, the cumulative energy demand (CED) associated with the production of each mortar mixture was quantified and reported.

Within the LCA model, SH was considered an agricultural residue. Following the cut-off allocation approach, no environmental burdens associated with rice cultivation, harvesting, or primary production were assigned to SH, since it is regarded as a by-product rather than the primary product of the agricultural system. In contrast, BSH was modeled as a product generated through the pyrolysis process. This process involves the thermochemical conversion of rice husk under oxygen-limited conditions and requires both electrical and thermal energy inputs, which were included in the life cycle inventory. Similarly, CSH was modeled by incorporating the calcination process into the production system. The thermal energy required to heat the SH and convert it into amorphous silica-rich ash was included in the inventory to accurately represent the environmental burdens associated with CSH production. For transparency, the main LCI assumptions are summarized as follows: functional unit, 1 m³ of mortar; system boundary, cradle-to-gate (raw material extraction, transportation, and mortar production); software, SimaPro; background database, Ecoinvent v3.4; impact assessment methods, the GHG Protocol (GWP100) and ReCiPe Endpoint (H/A), covering Human Health, Ecosystems, and Resources categories, alongside cumulative energy demand (CED); and allocation approach, a zero-burden (cut-off) allocation was applied to SH, consistent with its status as an agricultural residue.

3. Results

3.1. Flow table

The workability of the fresh mortar mixtures was evaluated using the Flow Table test in accordance with ASTM C230 [33], using a truncated cone mold (100 mm base diameter, 70 mm top diameter, 50 mm height) subjected to 25 drops within 15 seconds. The obtained flow spread values, expressed as a percentage increase in average base diameter, are presented in Fig. 4. The reference specimen (C₁₀₀) exhibited a flow spread of approximately 170 mm. With the incorporation of SH, the workability of the mortar was found to be dependent on the replacement level. At a replacement ratio of 3%, no significant change was observed compared to the reference mixture. However, increasing the replacement level to 5% resulted in an increase in flow spread to approximately 200 mm, indicating improved fluidity of the mixture. Nevertheless, at an 8% replacement level, the flow spread decreased to about 160 mm, becoming even lower than that of the reference specimen. This behavior can be attributed to the higher specific surface area and greater water absorption capacity of SH particles at elevated replacement levels, which reduce the amount of free water available in the mixture and consequently decrease workability. A similar trend was observed in mortars containing BSH. The incorporation of 3% biochar increased the flow spread to approximately 195 mm. However, at replacement levels of 5% and 8%, the flow spread stabilized at around 180 mm. The porous structure of biochar promotes the absorption of a portion of the mixing water; nevertheless, its surface roughness and particle morphology may improve particle distribution and fluidity at low replacement levels. At higher contents, the water absorption effect becomes dominant, limiting any further enhancement in workability. The highest flow spread values were obtained in mixtures containing CSH. Replacing 3% of cement with CSH increased the flow spread to approximately 235 mm, while replacement levels of 5% and 8% further increased it to about 250 mm. This remarkable improvement can be attributed to the fine particle size of the ash, its favorable particle size distribution, and the filler effect. The ultrafine particles of calcined rice husk ash fill the voids between cement grains, thereby reducing internal friction within the mixture and facilitating the movement of the cement paste. Furthermore, the relatively more uniform particle shape of CSH compared with SH and BSH plays a significant role in enhancing mortar fluidity.

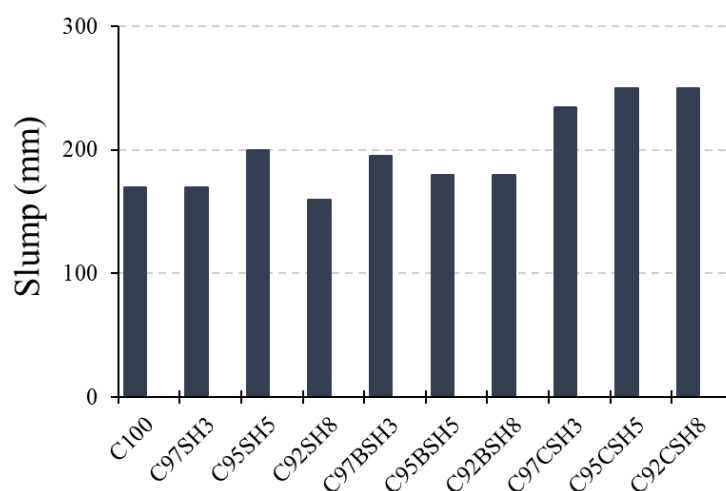


Fig. 4. Flow table test results.

Overall, in terms of workability performance, the materials can be ranked as follows: CSH > BSH > SH. Specifically, CSH increased the flow spread by approximately 38–47% compared to the control specimen, whereas BSH resulted in a modest increase of about 6–15%. In contrast, SH exhibited satisfactory performance only at the 5% replacement level, while an 8% replacement led to a reduction in workability. The Flow Table test results demonstrated that the processing method of SH has a decisive influence on mortar workability. While the use of SH and BSH provided only limited improvements in mixture fluidity, CSH significantly

enhanced the flow spread. This behavior is primarily attributed to the filler effect, improved particle size distribution, and reduced internal friction within the mixture. It was also observed that increasing the replacement level of SH beyond the optimum value reduced mortar workability due to its higher water absorption capacity and larger specific surface area. Overall, from the perspective of fresh-state performance, the incorporation of 5-8% CSH provided the most favorable workability among all investigated mixtures.

3.2. Compressive strength

Table 4 presents the mean and standard deviation (SD) of compressive strength for each mix design at 7, 14, and 28 days of curing, based on two specimens per condition. Fig. 5 presents the compressive strength development of mortar specimens incorporating SH, BSH, and CSH at curing ages of 7, 14, and 28 days. As expected, all mixtures exhibited a continuous increase in compressive strength with curing time due to the progressive hydration of cement and the formation of additional cementitious products. However, the rate of strength development and the ultimate compressive strength were significantly influenced by both the form of SH utilized and its replacement level. At the age of 7 days, most modified mixtures exhibited lower compressive strengths than the control specimen, indicating that the partial replacement of cement initially reduced the amount of readily available hydraulic binder. This effect was particularly pronounced for mixtures containing SH, where the incorporation of untreated lignocellulosic particles delayed strength development due to their limited pozzolanic reactivity and relatively high-water absorption capacity. Nevertheless, mixtures containing CSH showed comparatively higher early-age strengths, suggesting that the amorphous silica generated during the calcination process contributed to early pozzolanic reactions and enhanced particle packing. A substantial increase in compressive strength was observed between 7 and 14 days for all mixtures. This improvement can be attributed to the continuation of cement hydration as well as the gradual activation of supplementary cementitious reactions.

Table 4. Standard deviations of mortar compressive strength results.

Mix ID.	Compressive strength (MPa)		SD	AVG	Compressive strength (MPa)		SD	AVG	Compressive strength (MPa)		SD	AVG
	7-days	7-days			14-days	14-days			28-days	28-days		
C ₁₀₀	23.7	22.9	0.49	23.3	28.5	27.5	0.72	28.0	32.6	27.9	3.36	30.2
C ₉₇ SH ₃	8.4	9.0	0.42	8.7	16.8	17.6	0.58	17.2	23.4	24.5	0.74	23.9
C ₉₅ SH ₅	14.3	15.9	1.16	15.1	28.7	28.4	0.26	28.5	35.8	34.6	0.84	35.2
C ₉₂ SH ₈	14.8	16.2	0.94	15.5	35.5	34.0	1.06	34.8	40.7	39.9	0.58	40.3
C ₉₇ BSH ₃	13.5	13.9	0.28	13.7	30.2	32.1	1.32	31.1	41.7	40.5	0.84	41.1
C ₉₅ BSH ₅	15.4	13.1	1.62	14.3	29.0	30.1	0.78	29.5	35.7	37.2	1.02	37.9
C ₉₂ BSH ₈	14.1	13.2	0.66	13.7	28.8	27.9	0.66	28.4	36.7	35.6	0.78	36.2
C ₉₇ CSH ₃	20.4	17.5	2.04	18.9	32.0	30.8	0.88	31.4	44.1	42.8	0.88	43.4
C ₉₅ CSH ₅	13.8	13.5	0.27	13.7	25.9	25.2	0.44	25.5	36.8	33.4	2.37	35.1
C ₉₂ CSH ₈	11.9	12.6	0.44	12.3	19.0	20.0	0.71	19.5	30.9	28.1	1.93	29.6

The strength gain was particularly evident in mixtures containing higher proportions of SH and BSH, indicating that the internal curing effect provided by the porous structure of these materials may have facilitated further hydration. In addition, the fine particles generated during the biochar production process likely acted as micro-fillers, reducing pore connectivity and contributing to matrix densification. At 28 days, distinct performance trends emerged among the investigated materials. The mixture containing 3% calcined rice husk ash (C₉₇CSH₃) achieved the highest compressive strength among all specimens, reaching approximately 43.4 MPa. This represents a significant improvement compared with the control mixture and highlights the effectiveness of CSH as a supplementary cementitious material. The superior performance of this mixture is likely attributable to the combined effects of pozzolanic activity, filler action, and microstructural refinement, consistent with mechanisms widely reported for similar supplementary cementitious materials [12, 42]. It should be noted that the mechanistic explanations discussed throughout this section, including internal curing, secondary C-S-H formation, ITZ refinement, pore refinement, and crack-bridging effects, are proposed based on established mechanisms reported in the literature for comparable materials, and are not directly verified through microstructural characterization (e.g., SEM, XRD, or mercury intrusion porosimetry) in the present study; they should therefore be regarded as plausible interpretations consistent with the observed strength trends rather than directly confirmed mechanisms. The highly reactive amorphous silica present in the ash is expected to react with calcium hydroxide released during cement hydration, potentially producing additional calcium silicate hydrate (C-S-H) gel and enhancing matrix compactness.

The mixtures containing BSH also demonstrated notable improvements in compressive strength, particularly at the 3% replacement level. The best performance among the tested biochar levels, observed for C₉₇BSH₃, is consistent with the hypothesis that a limited amount of biochar can improve hydration efficiency through internal curing and moisture retention mechanisms, although this was not directly verified in the present study. However, increasing the biochar content beyond the optimum level resulted in a gradual reduction in strength. This behavior is likely associated with the porous nature of biochar, which may increase the overall void content and reduce the effective binder concentration when used at higher dosages. A different trend was observed for SH mixtures. Although the 3% replacement level resulted in relatively low compressive strength, increasing the replacement ratio led to a substantial improvement in later-age (up to 28 days) performance, with the 8% replacement mixture achieving one of the highest strengths among the tested specimens. This behavior may be associated with the delayed contribution of finely dispersed silica and a possible gradual improvement of the ITZ over time, although this was not directly examined. It is also possible that the

fibrous morphology of SH particles contributed to crack-bridging effects and stress redistribution within the hardened matrix; however, this proposed mechanism was not directly verified in this study. Overall, the results demonstrate that the processing method of SH plays a decisive role in determining the mechanical performance of cementitious composites. Among the investigated materials, CSH exhibited the highest efficiency in enhancing compressive strength due to its strong pozzolanic reactivity, while biochar provided beneficial internal curing effects at low replacement levels. The findings indicate that optimizing both the treatment method and replacement ratio of SH-derived materials is essential for maximizing the mechanical performance of sustainable cement-based composites.

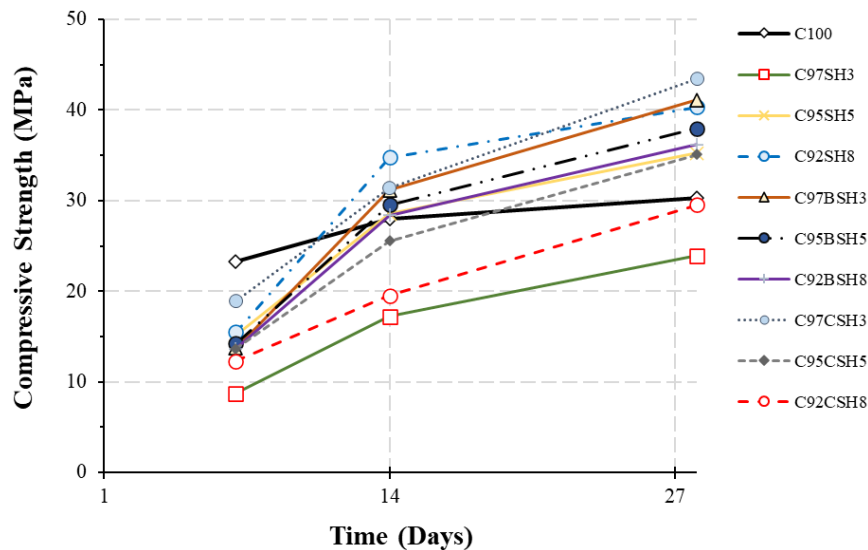


Fig. 5. Compressive strength of mortar samples.

Fig. 6 illustrates the normalized compressive strength of mortar specimens containing SH, BSH, and CSH relative to the control mixture at different curing ages. The normalization process enables a clearer comparison of the effectiveness of each replacement strategy independent of the absolute strength values. At the age of 7 days, all modified mixtures exhibited normalized strengths below 100%, indicating that the partial replacement of cement adversely affected early-age strength development. This reduction can primarily be attributed to the dilution effect caused by replacing a portion of the hydraulic binder with materials possessing lower initial reactivity. The most pronounced reduction was observed in mixture C₉₇SH₃, which retained only 37.4% of the compressive strength of the control specimen. Similarly, mixtures containing BSH and CSH exhibited normalized strengths ranging from approximately 52.8% to 81.3%. These results suggest that the beneficial effects associated with pozzolanic activity, micro-filler action, and internal curing had not yet fully developed during the first week of hydration. A markedly different trend emerged after 14 days of curing. Several mixtures surpassed the performance of the reference specimen, indicating the progressive activation of secondary strengthening mechanisms. The normalized strength of C₉₂SH₈ increased to 124.2%, while C₉₇BSH₃ and C₉₇CSH₃ achieved values of 111.2% and 112.1%, respectively. This substantial improvement reflects the continued hydration of cement and the increasing contribution of SH-derived materials to microstructural refinement. In particular, the porous structure of biochar may have promoted internal curing by gradually releasing absorbed water, thereby sustaining hydration reactions and reducing self-desiccation effects. Simultaneously, the fine particles present in CSH likely provided additional nucleation sites for hydration products, accelerating the formation of a denser cementitious matrix.

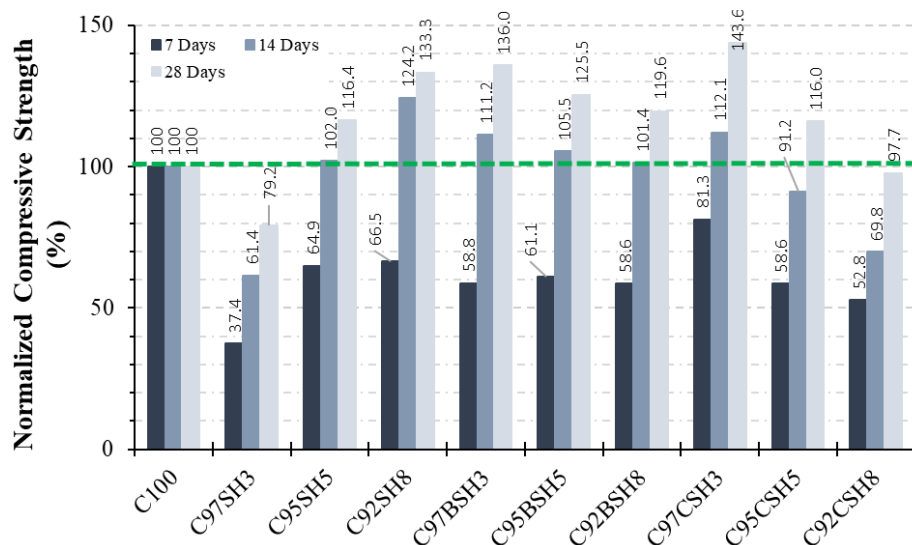


Fig. 6. Normalized compressive strength of mortar samples.

At 28 days, the superiority of selected mixtures became even more evident. The highest normalized compressive strength was recorded for C₉₇CSH₃, reaching 143.6% of the reference specimen's strength (i.e., a 43.6% increase). This remarkable enhancement

is consistent with a high pozzolanic efficiency of CSH when incorporated at moderate replacement levels. The reactive amorphous silica generated during calcination is expected to react with calcium hydroxide liberated from cement hydration, potentially producing additional secondary calcium silicate hydrate (C-S-H) gel and contributing to pore refinement, which would be consistent with the formation of a denser and mechanically stronger matrix; however, this mechanism was not directly verified through microstructural analysis in this study. Mixtures containing BSH also demonstrated favorable later-age performance (within the 28-day testing period). $C_{97}BSH_3$ and $C_{95}BSH_5$ achieved normalized strengths of 136.0% and 125.5%, respectively, while $C_{92}BSH_8$ maintained a value of 119.6%. These findings indicate that BSH can effectively enhance later-age strength when used within an appropriate dosage range. The observed improvements may be attributable to a combination of internal curing, enhanced hydration efficiency, and filler effects arising from the fine carbonaceous particles, consistent with mechanisms reported for similar carbon-based additives, although these were not directly measured in this study. Nevertheless, the gradual decline in performance with increasing replacement level suggests that excessive BSH contents introduce additional porosity and reduce the effective cement content, partially offsetting these beneficial mechanisms. The behavior of SH mixtures was particularly noteworthy. Although the incorporation of 3% SH resulted in substantial strength reductions throughout the curing period, increasing the replacement ratio to 5% and especially 8% produced significant later-age benefits. The normalized strength of $C_{92}SH_8$ reached 133.3% after 28 days, representing one of the highest values among all investigated mixtures. This result suggests that the influence of SH extends beyond simple cement replacement and may involve gradual microstructural enhancement, improved particle packing, and delayed pozzolanic interactions associated with the silica-rich composition of rice husk. In contrast, increasing the dosage of CSH beyond the optimum level resulted in a reduction in normalized strength. While $C_{97}CSH_3$ exhibited the highest performance, the normalized strength decreased to 116.0% and 97.7% for $C_{95}CSH_5$ and $C_{92}CSH_8$, respectively. This behavior indicates that the beneficial pozzolanic and filler effects become insufficient to compensate for the reduction in cement content at higher replacement ratios. Such a trend is characteristic of supplementary cementitious materials, where an optimum replacement level exists beyond which the dilution effect becomes dominant. Overall, the normalized strength analysis demonstrates that both the processing method and replacement level govern the mechanical efficiency of SH-derived materials. Among all investigated mixtures, the incorporation of 3% CSH produced the greatest enhancement in compressive strength, followed by 3% BSH and 8% SH. These findings confirm that suitable treatment of SH can transform an agricultural waste into an effective supplementary cementitious material capable of significantly improving the later-age mechanical performance of sustainable cement-based composites. Fig. 7 illustrates the typical failure pattern of mortar cube specimens under compressive loading. All specimens exhibited a typical shear-cone failure mode characteristic of cube compression tests, with diagonal cracking initiating near the loaded faces. No notable differences in failure mode were observed among the control and modified mixtures, suggesting that the incorporation of SH-derived materials did not alter the fundamental fracture mechanism, despite differences in load-carrying capacity.



Fig. 7. Failure of mortar samples under compressive test.

3.3. Compressive strength growth

Fig. 8 presents the contribution of different curing intervals (0-7, 7-14, and 14-28 days) to the overall 28-day compressive strength development of the mortar specimens. As expected, the reference mixture (C_{100}) exhibited rapid early-age strength development, with approximately 75.8% of its total 28-day strength gain achieved within the first seven days. The contribution of the subsequent curing intervals decreased considerably, accounting for only 16.7% and 7.5% during the 7-14 and 14-28 days periods, respectively. This behavior is characteristic of ordinary Portland cement, where the rapid hydration of clinker phases, particularly C_3S , governs early strength development. The incorporation of SH-derived materials significantly altered the kinetics of compressive strength development by shifting a larger proportion of strength gain from early ages to later curing periods.

Mortars containing SH exhibited a pronounced reduction in the contribution of the first seven days, while the proportions of strength development during the 7-14 and 14-28 days intervals increased markedly. This delayed strength development can primarily be attributed to the dilution effect caused by cement replacement and the limited early reactivity of untreated SH. Furthermore, the presence of lignocellulosic constituents in the SH may retard cement hydration by absorbing mixing water and interfering with the formation of early hydration products. A similar trend was observed for biochar-containing mixtures, although the redistribution of strength gain was more concentrated within the intermediate curing period. In particular, the 3% BSH mixture ($C_{97}BSH_3$) exhibited the lowest contribution from the first seven days (19.7%) and the highest contribution during the 7-14 days interval (56%). This behavior is associated with the porous microstructure and high specific surface area of BSH, which initially absorbs part of the mixing water, thereby slowing early hydration, before the gradual release of this stored water promotes internal curing during the intermediate stage, consistent with the mechanism discussed in Section 3.2.

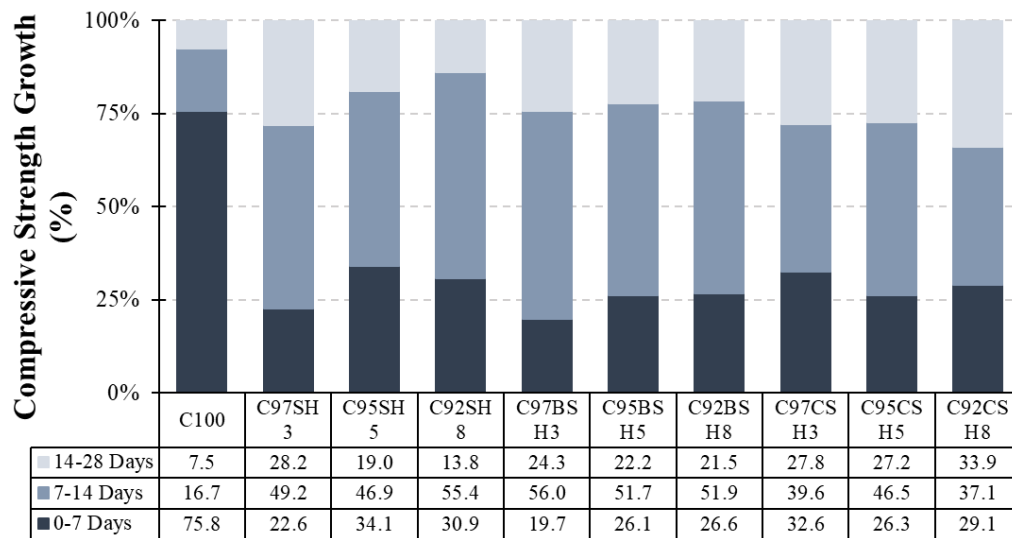


Fig. 8. Compressive strength growth of mortar samples.

Mortars incorporating CSH also demonstrated lower early-age strength contributions than the reference mixture; however, their strength development was more evenly distributed throughout the curing period. Notably, the contribution of the 14-28 days interval increased substantially, reaching 33.9% in the mixture containing 8% CSH ($C_{92}CSH_8$). This is consistent with the progressive pozzolanic reaction mechanism discussed in Section 3.2 for CSH. Overall, the results indicate that replacing cement with SH-derived materials modifies the mechanism of strength development rather than simply reducing or increasing compressive strength. While the reference mortar derives most of its strength during the first week, the modified mortars continue developing strength during later curing ages through a combination of ongoing cement hydration, internal curing effects (in the case of BSH), and secondary pozzolanic reactions (particularly in CSH mixtures). These findings suggest that evaluating the performance of SH-modified mortars solely based on early-age compressive strength may underestimate their strength development within the tested 28-day period; testing beyond 28 days would be required to substantiate any longer-term trajectory.

3.4. Environmental impact assessment results

An LCA was conducted to evaluate the environmental performance of mortar mixtures incorporating SH, BSH, and CSH at cement replacement levels of 3%, 5%, and 8%, using the reference mixture. The assessment considered GHG emissions, ReCiPe endpoint impact categories, and CED. These environmental analyses complement the mechanical performance results and highlight the sustainability potential of SH-derived supplementary cementitious materials.

The carbon footprint and GHG emissions of the different mortar mixtures per 1 m³ are presented in Fig. 9. The total GHG emissions, expressed as kg CO₂-equivalent, decreased with increasing replacement levels for both the SH and CSH mixtures, demonstrating the clear environmental benefits of partial cement substitution. The reference mixture exhibited the highest total emissions, reaching approximately 667.27 kg CO₂-eq/m³. In contrast, the 8% SH mixture ($C_{92}SH_8$) produced the lowest emissions, approximately 617 kg CO₂-eq/m³, followed by the 8% CSH mixture ($C_{92}CSH_8$) with approximately 623 kg CO₂-eq/m³. Mixtures containing BSH exhibited comparatively higher total GHG emissions, primarily because of the additional CO₂ emissions associated with the pyrolysis process and the carbon-rich nature of biochar production. These findings confirm that low-to-moderate cement replacement levels, particularly with CSH and SH, can effectively reduce the carbon footprint while maintaining or improving mechanical performance, thereby supporting circular economy principles in sustainable construction [43].

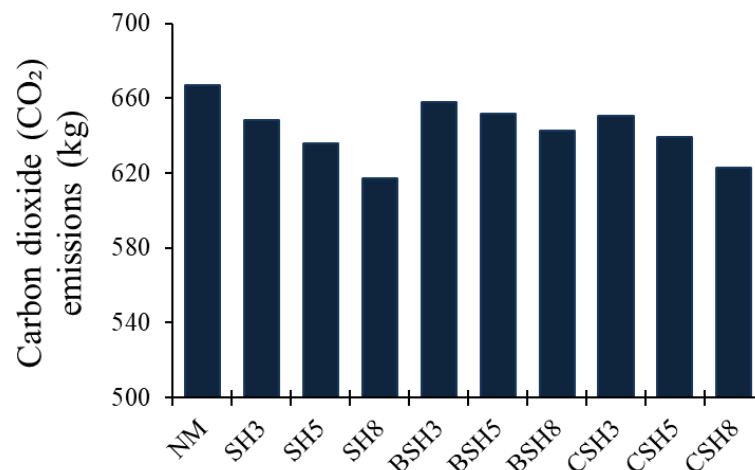


Fig. 9. Greenhouse gas (GHG)-based carbon dioxide emission results.

Similar trends were observed using the ReCiPe endpoint impact assessment method (Fig. 10), which aggregates environmental impacts into the three endpoint categories of Human Health, Ecosystems, and Resources. The control mixture exhibited the highest

overall environmental impact, with a total score of 6.22 points, mainly attributable to impacts on human health. All mixtures incorporating rice husk-derived materials demonstrated reductions in total endpoint scores, with the greatest improvements observed for the 8% SH and 8% CSH mixtures. Biochar-containing mixtures showed slightly higher overall endpoint scores because of increased ecosystem and resource impacts associated with the pyrolysis process. Nevertheless, these mixtures still offer significant environmental advantages through long-term carbon sequestration. Ecosystem damage decreased slightly in mixtures containing SH and CSH, while resource depletion followed a trend similar to that of GHG emissions owing to the reduced consumption of clinker and Portland cement. These findings are consistent with previous LCA studies on agricultural waste-derived SCMs, which have reported reductions in human toxicity, fossil resource depletion, and overall environmental burdens through partial cement replacement. Overall, the analysis demonstrates that SH-derived materials not only reduce direct GHG emissions but also provide broader environmental benefits across multiple endpoint categories [44].

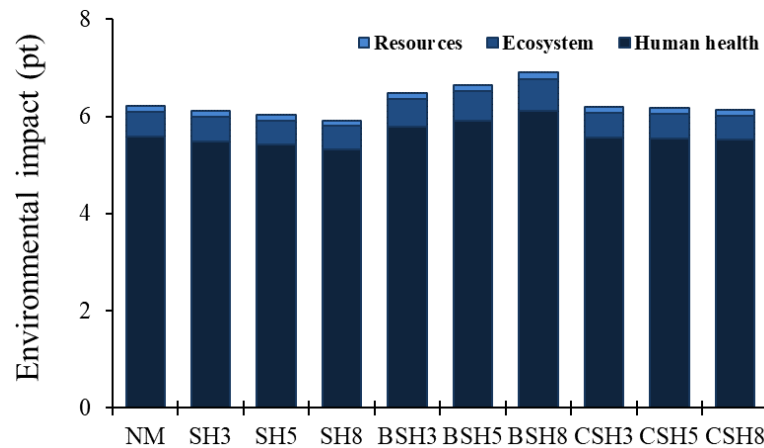


Fig. 10. Environmental endpoint impact categories of the investigated mixtures based on the ReCiPe.

The CED of the investigated mixtures is presented in Fig. 11. The lowest total fossil energy demand was obtained for mixtures containing higher replacement levels of SH and CSH, with $C_{92}SH_8$ and $C_{92}CSH_8$ requiring approximately 2795 MJ/m³ and 2864 MJ/m³, respectively, compared with 2996 MJ/m³ for the control mixture. This reduction is primarily attributed to the lower energy requirement associated with clinker production when cement is partially replaced by SH and CSH. In contrast, the BSH mixtures exhibited higher cumulative energy demands, reaching 3056 MJ/m³ for $C_{92}BSH_8$, mainly because of the non-renewable energy and biomass energy consumed during the pyrolysis process. Overall, the CED results further demonstrate the feasibility of utilizing SH-derived SCMs to reduce the embodied energy of cementitious mortars, with CSH and SH showing the greatest potential for sustainable construction applications [45].

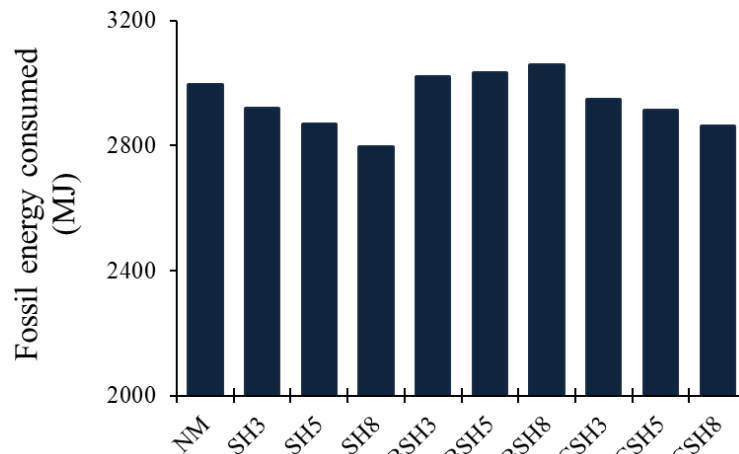


Fig. 11. Cumulative energy demand (CED) results of the investigated mixtures.

Overall, the LCA results indicate that replacing 8% of Portland cement with SH provided the greatest environmental benefits among all investigated mixtures. Compared with the reference mixture, $C_{92}SH_8$ reduced CO₂ emissions by 7.7%, the overall ReCiPe endpoint impact by 4.8%, and the cumulative energy demand by 6.7%. Likewise, the $C_{92}CSH_8$ mixture achieved reductions of 6.8%, 1.3%, and 4.4%, respectively. Although the $C_{92}BSH_8$ mixture reduced GHG emissions relative to the reference, its overall environmental performance was less favorable because of the additional energy consumption associated with the pyrolysis process.

A sensitivity analysis was performed on four key LCA assumptions: (I) the allocation approach applied to SH, (II) the pyrolysis energy input for BSH production, (III) the calcination energy input for ash production, and (IV) transport distances. The relative ranking of mixtures ($SH_8 > CSH_8 > BSH_8$, in order of decreasing environmental benefit) remained robust across all tested scenarios, although absolute GHG emissions and energy demand values were most sensitive to the pyrolysis and calcination energy inputs. The zero-allocation assumption for SH had a negligible effect (< 2%) on the overall results, since the primary environmental benefit derives from cement replacement rather than from the allocation method itself, while transport distance variations produced a moderate effect (3–5%). These findings support the robustness of the comparative conclusions drawn in this study, while absolute

values remain dependent on the underlying model assumptions. It should also be noted that background LCA data were sourced from global and rest-of-world (GLO/RoW) Ecoinvent datasets, including default "market for" transport distances based on global trade patterns; while Ecoinvent v3.4 provides robust and widely accepted background data, these datasets represent global average conditions rather than Iran-specific conditions, and a fully regionalized assessment would require site-specific inventory data. Finally, although the mechanical testing in this study did not extend beyond 28 days, biochar-amended mixtures are expected to provide an additional, longer-term environmental benefit through carbon sequestration within the biochar's stable carbon structure, independent of its mechanical performance.

3.5. Comparison with previous studies

To further evaluate the effectiveness of the proposed SH-derived supplementary cementitious materials, the 28-day compressive strength obtained in the present study was compared with representative results reported in previous investigations (Table 5). The comparison indicates that the mortar incorporating 3% CSH ($C_{97}CSH_3$) achieved the highest compressive strength (43.4 MPa) among all mixtures evaluated in the present work, followed closely by the 3% BSH mixture ($C_{97}BSH_3$) with a compressive strength of 41.1 MPa. Both mixtures exhibited substantial improvements over the control mortar (30.3 MPa), corresponding to strength increases of approximately 43.6% and 35.6%, respectively. Compared with previous studies employing agricultural or industrial waste materials as SCM, the proposed mixtures demonstrated competitive and, in several cases, superior mechanical performance.

Table 5. Comparison of 28-day compressive strength with previous studies.

References	Composition of powder materials	Compressive strength (MPa)
Current Study	100OPC	30.3
	97OPC + 3BSH	41.1
	97OPC + 3CSH	43.4
Alamian et al. [42]	100OPC	32.4
	65OPC + 20RC + 15GG	29.3
	65OPC + 25RC + 10GG	28.0
Ramezaniapour et al. [46]	100OPC	49.3
	70OPC + 30CC1	47.3
	70OPC + 20CC1 + 10LS	44.9
	70OPC + 30CC2	44.6
	70OPC + 20CC2 + 10LS	38.4
Dhandapani and Santhanam [47]	100OPC	62.1
	70OPC + 30FA	46.6
	55OPC + 30CC + 15LS	43.0
Mishra et al. [48]	100OPC	38.8
	55OPC + 30CC + 15LS	36.2

Note: OPC: ordinary Portland cement; BSH: rice husk biochar; CSH: calcined rice husk ash; RC: recycled calcined clay; GG: ground granulated blast furnace slag; CC1/CC2: calcined clay (type 1/type 2); LS: limestone; FA: fly ash. Subscripts denote replacement percentage by mass of binder.

Alamian et al. [42] reported compressive strengths ranging from 28.0 to 32.4 MPa for mortars incorporating recycled calcined clay and ground granulated blast furnace slag, indicating that relatively high replacement levels (30%) resulted in a noticeable reduction in strength due to the dilution effect. Similarly, the mortars developed by Mishra et al. [48] achieved a maximum compressive strength of 38.8 MPa, which remained lower than that obtained for both optimized mixtures in the present study. The results reported by Ramezaniapour et al. [46] and Dhandapani and Santhanam [47] reached higher absolute compressive strengths, with maximum values of 49.3 MPa and 62.1 MPa, respectively. However, these studies were conducted using different binder systems, replacement materials, mixture proportions, and water-to-binder ratios, making direct numerical comparisons inappropriate. More importantly, increasing the replacement ratio in those studies generally resulted in a gradual reduction in compressive strength, whereas the present study demonstrated that carefully optimized low replacement levels (3%) can not only compensate for the reduction associated with cement replacement but also produce a significant improvement in mechanical performance. The superior performance of the optimized mixtures can be attributed to the synergistic interaction of several mechanisms. In the case of CSH, the high content of amorphous silica promotes secondary pozzolanic reactions, potentially leading to additional C-S-H formation, pore refinement, and a denser cementitious matrix. Meanwhile, BSH may contribute through internal curing, enhanced hydration, and heterogeneous nucleation of hydration products. These proposed mechanisms, consistent with those reported for comparable materials in the literature (though not directly verified in this study through microstructural characterization), may collectively help explain why the best-performing mixtures outperformed many previously reported sustainable cementitious composites. Overall, the comparison presented in Table 5 demonstrates that the strategy adopted in this study, combining low replacement levels with appropriate SH processing techniques, is more effective than simply increasing the replacement ratio. The findings suggest that optimizing the physicochemical characteristics of SH-derived materials is more critical than maximizing their dosage, providing an efficient pathway for producing sustainable cement-based materials with enhanced mechanical performance.

4. Conclusions

This study evaluated the effects of SH, BSH, and CSH as partial cement replacement materials on the fresh and mechanical properties of mortar. The results demonstrate that the processing method of SH plays a decisive role in determining the performance of cementitious composites by influencing both hydration behavior and microstructural development.

- Among the investigated materials, CSH exhibited the most beneficial overall performance. Its incorporation significantly improved mortar flowability owing to its fine particle size and filler effect, while also producing the highest 28-day compressive strength. The mortar containing 3% CSH ($C_{97}CSH_3$) achieved approximately 143.6% of the reference mixture's compressive strength (i.e., a 43.6% increase), consistent with the combined effects of pozzolanic reactivity, nucleation, particle packing, and pore refinement, which are proposed as plausible contributors to the observed mechanical performance, although not directly verified in this study.
- Although all SH-derived materials reduced the proportion of early-age strength development compared with the reference mixture, they promoted a more sustained strength gain at later curing ages. This finding confirms that replacing cement with SH-derived materials modifies the hydration kinetics and shifts strength development from rapid early hydration toward progressive reactions at later ages (within 28 days). The contributions of secondary pozzolanic reactions, continued hydration, and, in the case of BSH, internal curing is proposed to become increasingly significant as curing progresses, consistent with mechanisms reported for similar materials, though not directly measured here.
- Biochar proved to be an effective supplementary cementitious material at low replacement levels. In particular, the mixture containing 3% BSH ($C_{97}BSH_3$) exhibited favorable later-age mechanical performance (within 28 days), suggesting that a limited amount of BSH can improve hydration efficiency through internal curing while promoting a denser cementitious matrix. However, increasing the BSH dosage beyond the optimum level diminished these benefits because of the dilution effect and increased porosity.
- The SH mixtures exhibited the slowest early-age strength development but continued to gain strength throughout the curing period. Notably, the mixture containing 8% SH ($C_{92}SH_8$) showed one of the greatest late-age strength growth rates, demonstrating that untreated SH can contribute to continued strength development at later ages within the tested 28-day period. Nevertheless, its overall mechanical performance remained inferior to that of the best-performing CSH mixture ($C_{97}CSH_3$) among the tested levels.
- The LCA demonstrated that incorporating SH-derived materials substantially improved the environmental performance of mortar by reducing GHG emissions, CED, and overall environmental impacts. Among the investigated mixtures, the 8% SH mixture ($C_{92}SH_8$) exhibited the greatest environmental benefits, achieving reductions of approximately 7.7% in CO_2 emissions, 4.8% in the ReCiPe endpoint score, and 6.7% in CED compared with the reference mixture. Similarly, the 8% CSH mixture ($C_{92}CSH_8$) also showed considerable environmental improvements, whereas the BSH mixtures exhibited comparatively higher embodied impacts because of the additional energy required during the pyrolysis process. Nevertheless, BSH remains an attractive low-carbon supplementary cementitious material owing to its long-term carbon sequestration potential and favorable mechanical performance at low replacement levels.

Overall, the findings demonstrate that SH-derived materials offer an effective strategy for simultaneously improving the sustainability and engineering performance of cementitious mortars. Among the investigated mixtures, $C_{97}CSH_3$ delivered the best overall mechanical performance, whereas $C_{92}SH_8$ achieved the greatest reduction in environmental impacts. Consequently, the optimum replacement level should be selected according to the primary design objective, prioritizing low CSH contents for mechanical performance and higher SH contents for environmental sustainability. These results provide a practical basis for designing low-carbon cementitious materials with reduced clinker consumption while maintaining satisfactory engineering performance. The findings should, however, be interpreted in light of two limitations. First, compressive strength was determined from two specimens per mixture at each curing age, limiting the statistical robustness of the comparisons. Second, the proposed mechanisms are supported by established literature rather than direct experimental evidence, as no microstructural characterization (XRD, SEM, BET/Blaine surface area, TGA, and MIP/porosity) or particle size analysis was performed. Future research should incorporate comprehensive microstructural analyses and a larger number of test specimens to validate the proposed mechanisms and enable more rigorous statistical evaluation. In addition, a constant water-to-binder ratio (0.42) was applied across all mixtures without correction for the water absorption of the rice husk-derived materials or pre-saturation of these materials prior to mixing. As a result, the effective water available for cement hydration may have varied between mixtures in an uncontrolled manner, which should be considered when interpreting the reported strength differences. Future studies should incorporate water absorption measurement and corresponding mixing water adjustment (or material pre-saturation) to isolate this effect.

Statements & Declarations

Author contributions

Lobat Hosseinzadeh: Conceptualization, Idea, Methodology, Investigation, Visualization, Writing - Original Draft, Formal analysis.

Morteza Jamshidi: Conceptualization, Methodology, Supervision, Project administration, Validation, Writing - Review & Editing.

Mehdi Dehestani: Conceptualization, Methodology, Supervision, Project administration, Validation, Writing - Review & Editing.

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Declarations

The data presented in this study will be available on interested request from the corresponding author.

Data availability

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Numerical Investigation of Reinforced Concrete Deep Beams with Square Web Openings: Influence of Opening Configuration and Reinforcement Detailing

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ABSTRACT

Web openings are frequently introduced into reinforced concrete deep beams to accommodate utility services; however, these discontinuities may significantly disrupt the internal load-transfer mechanism and adversely affect structural performance. This study investigates the influence of square opening size and location on the behavior of reinforced concrete deep beams using nonlinear finite element analysis in ABAQUS. Unlike previous studies that primarily focused on individual parameters such as opening geometry, opening location, or strengthening techniques, the present study systematically investigates the combined effects of square opening size, vertical opening location, and reinforcement detailing on the structural behavior of reinforced concrete deep beams. The numerical model was developed based on the concrete damaged plasticity (CDP) approach and validated against previously published experimental results, showing good agreement in terms of load-displacement response, crack patterns, and ultimate load capacity. Six deep beam specimens containing square web openings with side lengths of 200 and 250 mm positioned at various levels within the shear span were analyzed. The effects of opening configuration on crack propagation, principal stress distribution, and load-carrying capacity were evaluated. The results showed that openings located at the mid-depth of the web produced the greatest disturbance to the main compression strut, leading to increased stress concentration, extensive diagonal cracking, and reduced structural capacity. Increasing the opening size further intensified these effects by disrupting a larger portion of the principal compression strut. To mitigate these adverse effects, a modified reinforcement arrangement was proposed by redistributing web reinforcement around the opening and adding confinement reinforcement in critical regions. This detailing increased the ultimate load capacity by approximately 3-10%, delayed crack initiation by up to 44%, and improved principal stress redistribution. Overall, the proposed reinforcement arrangement effectively enhanced the structural behavior of deep beams with square web openings and provides practical guidance for reinforcement design in disturbed regions.

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1. Introduction

Reinforced concrete deep beams constitute a distinct class of structural members that are widely used in transfer girders, pile caps, foundation components, and coupled wall systems. Unlike slender beams, deep beams exhibit a highly nonlinear strain

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distribution over the depth of the section. Consequently, the Bernoulli hypothesis is no longer applicable, and their behavior cannot be adequately described using classical flexural theory. The theoretical basis of this behavior originates from Saint-Venant's principle, which explains the formation of disturbed regions (D-regions) and the disruption of stress trajectories in the vicinity of load application points and supports. Based on this concept, the strut-and-tie model (STM) was developed as an efficient approach for the analysis and design of deep beams [1] and was subsequently incorporated into major international design codes, including ACI 318 [2]. Within this framework, concrete compressive fields (struts) and steel tensile members (ties) interact through nodal zones to establish the internal load-transfer mechanism of the member.

Nevertheless, in practical applications, the provision of web openings is often unavoidable due to architectural requirements and the need to accommodate mechanical and electrical services. Among the various opening configurations, rectangular (including square) openings are the most commonly adopted because of their ease of construction and compatibility with utility ducts [3]. The introduction of such openings significantly disrupts the ideal load-transfer mechanism represented by the strut-and-tie model [4-9]. When a rectangular opening intersects the natural compressive struts within the shear region, severe stress concentrations develop at its sharp corners, resulting in a substantial reduction in stiffness, shear capacity, and ultimate strength of the member. Furthermore, the initiation and propagation of cracks are markedly altered, leading to a more pronounced and aggressive deterioration of the structural response.

Early investigations by Ray [10] demonstrated that large web openings located within the load-transfer zone between the applied load and the support can significantly reduce the ultimate load-carrying capacity of deep beams by disrupting the internal force-transfer mechanism. Similarly, El Maaddawy and Sherif [4] conducted experimental studies on reinforced concrete deep beams with openings and proposed empirical relationships for predicting shear strength. Their findings highlighted that the structural response is primarily governed by the extent of disturbance imposed on the natural stress flow paths. Regarding the influence of opening geometry, Kumari et al. [11] reported that rectangular and square openings produce substantially greater reductions in shear capacity than circular openings due to stress concentration effects at their sharp corners, resulting in a more brittle structural response. Mohamed et al. [12], through numerical analyses of deep beams with rectangular openings, recommended that openings should not interrupt the predicted compressive struts and that their depth should not exceed 20% of the overall beam depth. Furthermore, Ali et al. [13] demonstrated that both the opening aspect ratio and the reinforcement arrangement have a significant influence on strain distribution and the overall structural performance of deep beams.

Recent numerical and experimental investigations have provided further insight into the behavior of deep beams containing web openings. Advanced finite element analyses employing concrete damaged plasticity (CDP) models have confirmed that web openings significantly reduce stiffness and load-carrying capacity by interrupting the principal compressive stress trajectories within the beam. These studies consistently identify opening location as one of the most influential parameters governing structural performance, with the greatest reduction in strength occurring when the opening intersects the direct load-transfer path between the loading point and the support [14, 15]. Concerning opening geometry, several researchers have reported that square openings generally exhibit poorer structural performance than circular openings because of the severe stress concentrations developed at sharp corners [14, 15]. However, Al-Mahbashi et al. [16] suggested that when identical reinforcement interruption conditions are considered, the influence of opening shape on ultimate capacity may become less pronounced. Recent investigations have also indicated that increasing the opening-height-to-beam-depth ratio, particularly beyond approximately 0.33, may promote premature shear failure and alter the governing failure mechanism of deep beams. These findings collectively emphasize the importance of considering opening size, opening location, and reinforcement detailing when evaluating the structural response of deep beams containing web openings.

Despite the extensive body of research on deep beams with web openings, important gaps still remain in the available literature. Although several analytical and empirical approaches have been proposed, a reliable and widely accepted method for predicting the load-carrying capacity of deep beams with web openings is still lacking. Moreover, several recent numerical studies have relied on the assumption of perfect bond between internal components, thereby neglecting the potential influence of debonding mechanisms on structural response [15, 16]. In addition, many investigations have been based on relatively limited sample sizes and have focused primarily on static loading conditions. Although considerable attention has been devoted to comparing opening shapes and evaluating strengthening techniques, the combined influence of opening dimensions, vertical opening location, and reinforcement arrangement has not been systematically investigated. In particular, a need remains for detailed parametric studies addressing different vertical opening positions and their interaction with opening size in disturbed regions [14, 16]. The present study seeks to address this gap through a comprehensive finite element investigation of reinforced concrete deep beams containing square web openings of different sizes and vertical locations.

Accordingly, the present study investigates the nonlinear behavior of simply supported reinforced concrete deep beams containing web openings using the finite element method (FEM). A series of beams with identical overall dimensions subjected to concentrated loading were analyzed, incorporating openings of different sizes positioned at various vertical locations within the web. The primary variables considered in this study are opening size, opening location, and reinforcement arrangement around the opening region. Structural performance is evaluated in terms of ultimate load capacity, cracking load, crack propagation pattern, and principal stress distribution. The numerical model is first validated against available experimental results and subsequently employed in a comprehensive parametric investigation. Following the initial parametric investigation, a modified reinforcement arrangement was proposed around the openings, and its effectiveness in improving structural performance was evaluated.

Despite the considerable body of research on reinforced concrete deep beams with web openings, the combined influence of

square opening size, vertical opening location, and reinforcement detailing has not yet been systematically investigated within a unified numerical framework. Previous studies have generally focused on individual parameters, such as opening geometry, opening location, or strengthening techniques, without comprehensively examining their interaction and combined effect on load-transfer mechanisms within disturbed regions (D-regions).

To address this research gap, the present study provides a comprehensive finite element investigation of reinforced concrete deep beams containing square web openings. The scientific significance of this study lies in clarifying the combined influence of opening configuration and reinforcement detailing on crack development, principal stress redistribution, and structural performance. Furthermore, a practical reinforcement rearrangement strategy is proposed and evaluated to improve the structural behavior of deep beams with square web openings while requiring only a modest increase in reinforcement. The findings are expected to contribute to a better understanding of force-transfer mechanisms in disturbed regions and to provide practical guidance for the design of reinforced concrete deep beams containing service openings.

2. Finite element modeling and validation

2.1. Structural behavior of deep beams and numerical modeling approach

Reinforced concrete deep beams exhibit structural behavior significantly different from that of conventional slender beams due to their relatively small span-to-depth ratio. In such members, the assumption of a linear strain distribution across the section depth is no longer valid [17], and load transfer occurs primarily through compressive stress trajectories and discontinuity regions (D-regions) [2]. Consequently, conventional beam analysis methods based on classical flexural theory are unable to accurately represent the actual behavior of deep beams.

Several analytical approaches have been developed for the analysis of deep beams, including elasticity theory, the strut-and-tie model (STM), and the finite element method (FEM). Elasticity-based approaches are generally suitable for evaluating the pre-cracking behavior of deep beams; however, their applicability is limited in predicting the ultimate response of the member due to the assumption of linear material behavior. In contrast, the strut-and-tie model, which has been adopted by major design codes such as ACI 318, idealizes load transfer through an equivalent truss system composed of concrete compressive struts and steel tensile ties [17]. Although STM has proven to be an effective design tool [18–24], the determination of load paths and the configuration of struts largely depends on engineering judgment and may involve considerable uncertainty when applied to structural members with complex geometries, particularly those containing web openings.

The presence of openings in deep beams disrupts the natural load-transfer mechanism, alters or interrupts the concrete compressive struts, and generates significant stress concentrations around the opening boundaries. Under such circumstances, the finite element method (FEM) provides a more reliable framework for investigating the structural response due to its ability to simulate nonlinear material behavior, stress redistribution, crack initiation and propagation, and the interaction between concrete and reinforcing steel. Therefore, ABAQUS [25] was employed in the present study to perform nonlinear finite element analyses of reinforced concrete deep beams containing web openings.

2.2. Experimental specimens

To validate the accuracy of the developed finite element model, six reinforced concrete deep beams tested experimentally by El Maaddawy and Sherif [4] were numerically simulated using ABAQUS [25]. The numerical results obtained from the present study were subsequently compared with both the experimental data and the finite element results reported by Hawileh et al. [6].

The investigated specimens had overall dimensions of 1200 mm × 500 mm × 80 mm and contained square web openings measuring either 200 mm × 200 mm or 250 mm × 250 mm. The openings were positioned at different locations within the shear span to produce varying degrees of disturbance in the load-transfer mechanism and the concrete compressive struts. The geometric characteristics, opening configurations, and specimen designations are summarized in Table 1 and illustrated in Fig. 1.

For all specimens, the concrete compressive strength was taken as 21 MPa. Longitudinal reinforcement and web reinforcement were modeled using S420 and S300 steel grades, respectively. Detailed reinforcement layouts of the tested beams are illustrated in Fig. 2.

Table 1. Characteristics of the studied deep beam specimens.

Specimen	Opening location	Opening size (mm)
NS-200-T	Top of shear span	200 × 200
NS-250-T	Top of shear span	250 × 250
NS-200-C	Middle of shear span	200 × 200
NS-250-C	Middle of shear span	250 × 250
NS-200-B	Bottom of shear span	200 × 200
NS-250-B	Bottom of shear span	250 × 250

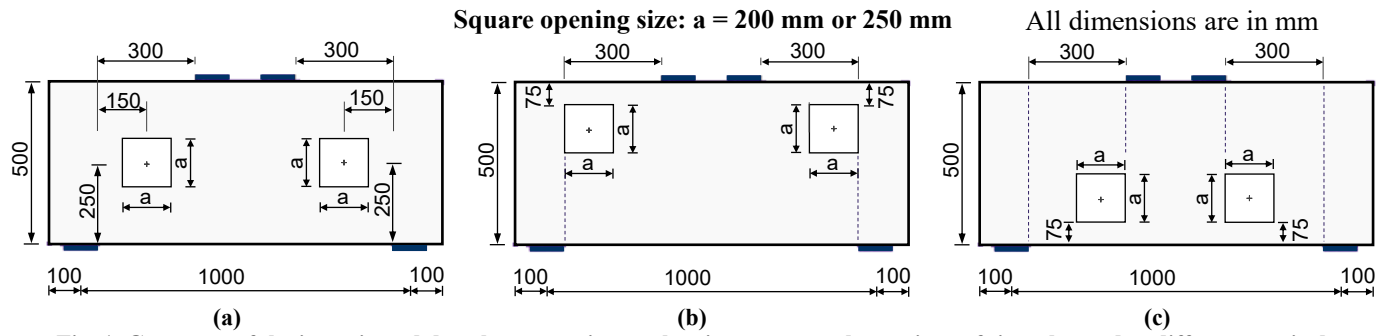


Fig. 1. Geometry of the investigated deep beam specimens showing square web openings of size a located at different vertical positions within the shear span: (a) center; (b) top; and (c) bottom [4].

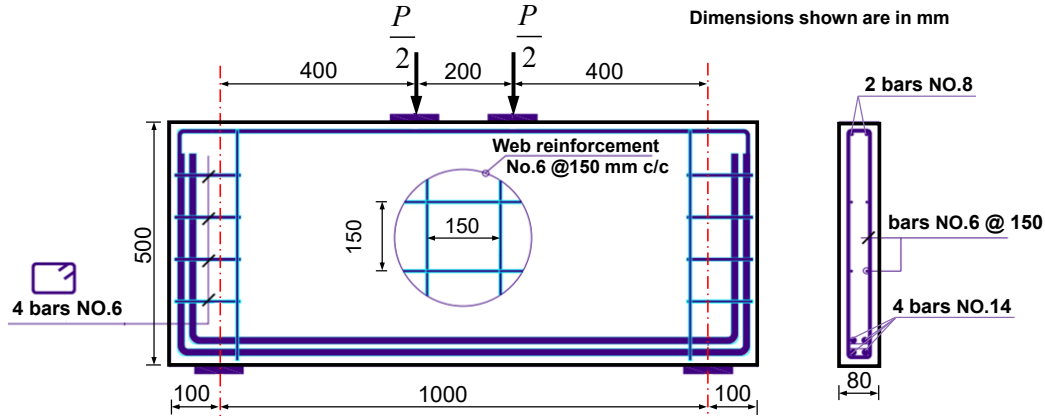


Fig. 2. Geometrical dimensions, reinforcement layout, loading arrangement, and cross-sectional details of the experimentally tested deep beam specimen used for model validation [4].

2.3. Material constitutive models

2.3.1. Concrete behavior under uniaxial compression and tension

To simulate the nonlinear compressive behavior of concrete, the Hognestad stress-strain model [26] was adopted. The Hognestad model was selected because it provides a realistic representation of the nonlinear compressive behavior of normal-strength concrete while maintaining a relatively simple mathematical formulation. Owing to its proven accuracy and extensive use in numerical studies of reinforced concrete structures, it is considered an appropriate constitutive model for the present investigation. This model consists of a parabolic ascending branch up to the peak compressive strength, a peak point corresponding to a compressive strain of 0.002, and a descending softening branch in which the stress gradually decreases with increasing strain. In the present study, the ultimate concrete strain was taken as 0.0038, and the post-peak response was represented by a linear descending branch.

For modeling the tensile behavior of concrete, a simplified stress-strain relationship was employed. In this model, the tensile stress increases linearly up to the tensile strength of concrete, followed by a sudden stress drop to 60% of the peak tensile strength. Thereafter, the stress decreases linearly until it reaches zero. The compressive and tensile constitutive relationships adopted for concrete are illustrated in Fig. 3 [6].

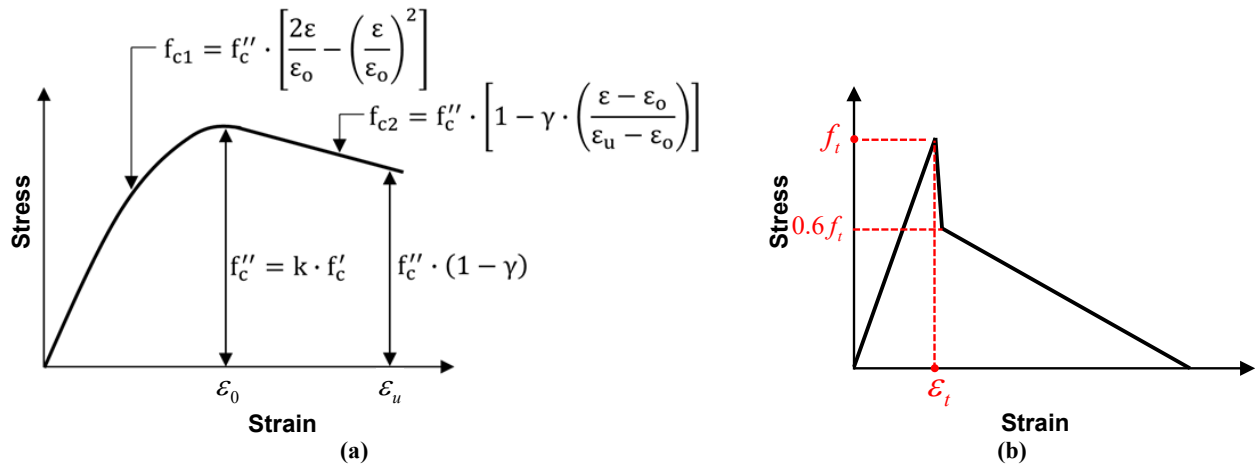


Fig. 3. Concrete stress-strain relationships adopted in the numerical model: (a) uniaxial compressive; and (b) Uniaxial Tensile.

2.3.2. Reinforcing steel constitutive model

The reinforcing steel was modeled using a bilinear elastic-plastic constitutive law incorporating strain hardening. This constitutive model was adopted because it accurately reproduces the yielding and post-yield hardening behavior of conventional

reinforcing bars while remaining computationally efficient and numerically stable in nonlinear finite element analyses. The initial portion of the curve represents linear elastic behavior, while beyond the yield point the material response continues with a strain-hardening slope equal to 2% of the initial elastic modulus [27]. The modulus of elasticity and Poisson's ratio of the reinforcing steel were assumed to be 200 GPa and 0.30, respectively [2]. The stress-strain relationships adopted for S300 and S420 reinforcing steels are presented in Fig. 4.

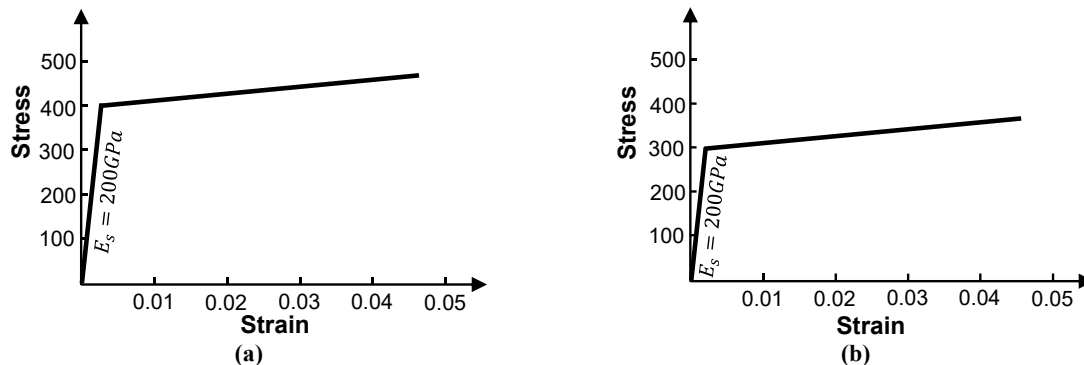


Fig. 4. Stress-strain relationships adopted for reinforcing steel modeling: (a) grade 420 steel; and (b) grade 300 steel.

2.3.3. Concrete damaged plasticity (CDP) model

To capture the multiaxial behavior of concrete, the concrete damaged plasticity (CDP) model available in ABAQUS [25] was employed. This constitutive model combines the principles of plasticity theory and continuum damage mechanics, enabling the simulation of tensile cracking, compressive crushing, progressive stiffness degradation, and post-peak material softening within a unified framework.

Compared with alternative constitutive models available in ABAQUS [25], the concrete damaged plasticity (CDP) model provides a comprehensive framework for simulating the nonlinear behavior of reinforced concrete under complex multiaxial stress states. The model is capable of representing tensile cracking, compressive crushing, stiffness degradation, damage evolution, and the distinct responses of concrete under tension and compression through independent damage variables. These capabilities are particularly important for the present study, in which the presence of square web openings results in severe stress redistribution, localized cracking, and progressive deterioration of the load-transfer mechanism within disturbed regions (D-regions). Furthermore, the CDP model has been extensively validated and widely adopted in numerical investigations of reinforced concrete structures, demonstrating both reliable predictive capability and numerical robustness. For these reasons, the CDP model was selected as the constitutive model for concrete throughout the present finite element analyses.

2.4. Finite element model

2.4.1. Element types

The numerical simulations were performed using a three-dimensional finite element model developed in ABAQUS [25]. Concrete was modeled using eight-node linear brick elements with reduced integration (C3D8R). These elements have been widely employed in nonlinear analyses of reinforced concrete structures due to their computational efficiency and their ability to alleviate shear locking problems while maintaining satisfactory accuracy.

The reinforcing bars were modeled using two-node truss elements (T3D2), which are capable of transmitting only axial forces and therefore provide an appropriate representation of the mechanical behavior of reinforcing steel embedded in concrete members.

2.4.2. Boundary conditions and loading configuration

The validated beam specimens were modeled as simply supported members with an effective span of 1000 mm., a width of 80 mm, and a depth of 500 mm, with an effective span of 1000 mm. All specimens were subjected to four-point bending loading conditions. The beams were modeled as simply supported members, and two concentrated loads were applied symmetrically on the top surface of the beam at a distance of 400 mm from each support.

This loading arrangement generated significant shear stresses within the shear spans, where the web openings were located, thereby providing suitable conditions for investigating the influence of openings on the load-transfer mechanism, stress distribution, and structural performance of reinforced concrete deep beams.

A sufficiently refined finite element mesh was adopted to accurately capture the stress concentration around the web openings while maintaining computational efficiency. The nonlinear analyses were performed using the ABAQUS [25] implicit solver with automatic load incrementation until convergence was achieved at each increment.

2.5. Model validation

2.5.1. Load-displacement response

To assess the accuracy of the developed finite element model, the load-displacement responses obtained from the numerical

simulations were compared with the experimental results reported by El Maaddawy and Sherif [4], as well as the numerical results presented by Hawileh et al. [6]. Fig. 5 illustrates the comparison of the load-displacement curves for the investigated specimens.

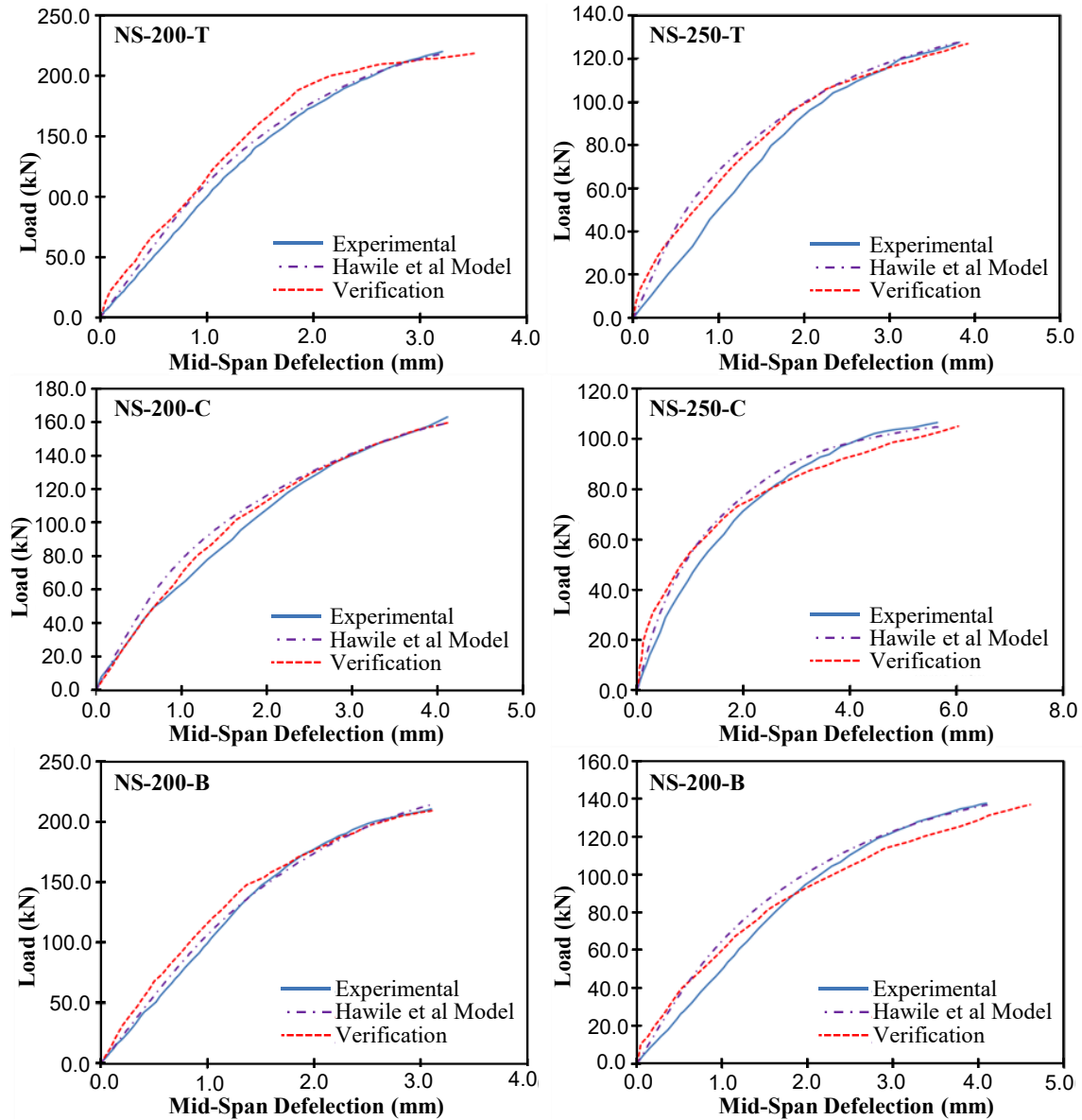


Fig. 5. Comparison of midspan load-displacement curves for the validation specimens obtained from experimental tests, previous numerical studies, and the present FE model.

The results demonstrate that the proposed model is capable of accurately reproducing the initial stiffness, nonlinear response, cracking load, and ultimate load-carrying capacity of the deep beams. As illustrated in Fig. 5, the numerical load-displacement curves show good agreement with the experimental responses throughout the loading history. This visual agreement, together with the close correspondence of the crack propagation patterns and the quantitative comparison of the P_{cr} / P_u , confirms the reliability of the developed finite element model for simulating the structural behavior of reinforced concrete deep beams containing web openings.

2.5.2. Crack patterns and failure mechanisms

To evaluate the capability of the numerical model in predicting crack initiation and propagation, the crack patterns obtained from the finite element analyses were compared with the corresponding experimental observations and the numerical results reported by Hawileh et al. [6]. The comparisons for specimens NS-200-T, NS-200-B, and NS-250-C are presented in Fig. 6. The numerical model accurately predicted both the crack initiation locations and the principal crack propagation paths observed in the experimental specimens. In all investigated beams, diagonal cracks originated from the corners of the square openings and propagated toward the loading and support regions, demonstrating a high level of agreement with the experimental observations.

2.5.3. Quantitative validation based on the cracking load ratio (P_{cr} / P_u)

To further evaluate the predictive capability of the developed finite element model, a quantitative validation was performed using the ratio of cracking load to ultimate load (P_{cr} / P_u). This parameter provides a meaningful indicator of the onset of cracking relative to the ultimate structural capacity and reflects the ability of the numerical model to capture the transition from the elastic stage to the nonlinear response of reinforced concrete deep beams.

The experimental results reported by El Maaddawy and Sherif [4] indicate that the cracking load occurred at approximately 31–51% of the ultimate load capacity for the investigated specimens. The corresponding values predicted by the present finite element model ranged from 27% to 52%. The close overlap between these ranges demonstrates that the numerical model provides a reliable prediction of crack initiation and the subsequent stiffness degradation associated with the development of diagonal cracking.

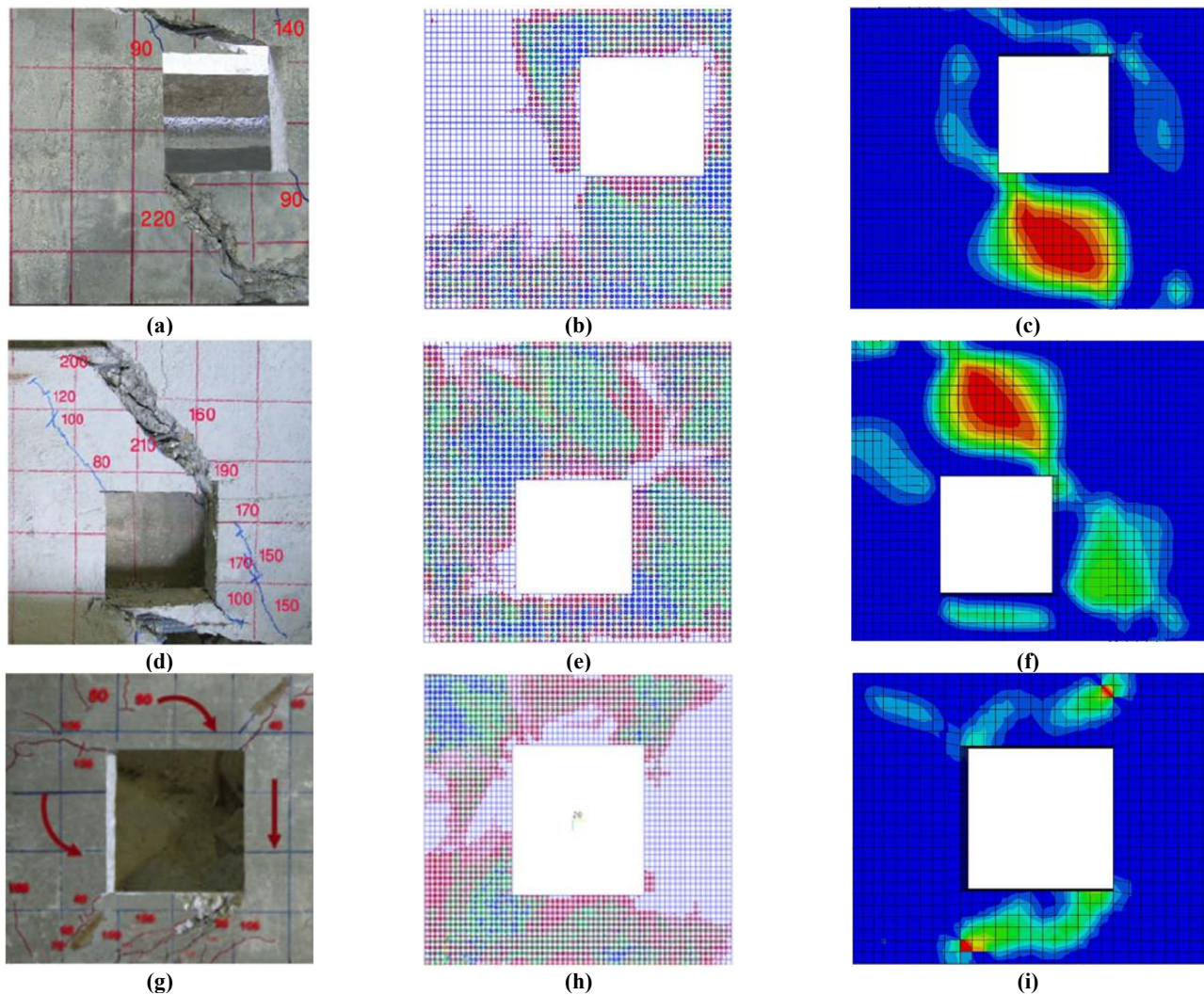


Fig. 6. Comparison of experimentally observed and numerically predicted crack patterns for validation specimens NS-250-C, NS-200-B, and NS-200-T: (a, d, and g) experimental results; (b, e, and h) reference numerical results; and (c, f, and i) predictions of the present finite element model.

3. Results and discussion

3.1. Studied specimens

To investigate the influence of square web openings on the structural behavior of reinforced concrete deep beams, six numerical specimens were analyzed using nonlinear finite element analysis (FEA). The specimens were categorized according to two geometric parameters: (I) opening size (200 mm and 250 mm) and (II) vertical opening location within the web (center, top, and bottom). These opening locations were intentionally selected to produce different levels of interference with the principal compression strut that transfers load from the loading point to the support.

It should be noted that the geometrical configuration, material properties, and reinforcement details of the investigated specimens are identical to those used in the validation study presented in Section 2. Therefore, the validated numerical models were adopted as the reference specimens in the present parametric investigation. To establish a consistent and scalable nomenclature suitable for future studies involving different opening geometries, a new specimen designation system was introduced. In this notation, SO denotes a square opening, the following number indicates the opening size (mm), and the final letter identifies the opening location within the beam depth (C = Center, T = Top, and B = Bottom). Furthermore, specimens with modified reinforcement arrangements are identified by the suffix –M. The selected specimens represent all possible combinations of the investigated opening sizes and vertical opening locations considered in this study.

Accordingly, the six reference specimens investigated in this study are designated as SO-200C, SO-200T, SO-200B, SO-250C, SO-250T, and SO-250B. The corresponding specimens with modified reinforcement layouts are denoted as SO-200C-M, SO-200T-M, SO-200B-M, SO-250C-M, SO-250T-M, and SO-250B-M, respectively.

The parametric investigation focuses on evaluating the effects of opening size and vertical opening location on the structural

response of deep beams and assessing the effectiveness of a modified reinforcement arrangement around the openings. In addition, a reinforcement modification strategy is proposed and assessed to mitigate the adverse effects of geometric discontinuities introduced by the openings. The effectiveness of the proposed reinforcement arrangement is subsequently evaluated through comparisons of load-displacement responses, cracking patterns, and stress-field distributions.

3.2. Crack patterns and failure mechanisms

Fig. 7 illustrates the initiation and propagation patterns of cracks in deep beams with web openings obtained from the finite element analyses, presented in the form of tensile plastic strain contours. Examination of the results indicates that both the crack initiation locations and propagation paths are strongly influenced by the size and vertical position of the openings. In all specimens, the corners of the square openings acted as critical regions for the initiation of diagonal shear cracks due to geometric discontinuities and the associated stress concentrations.

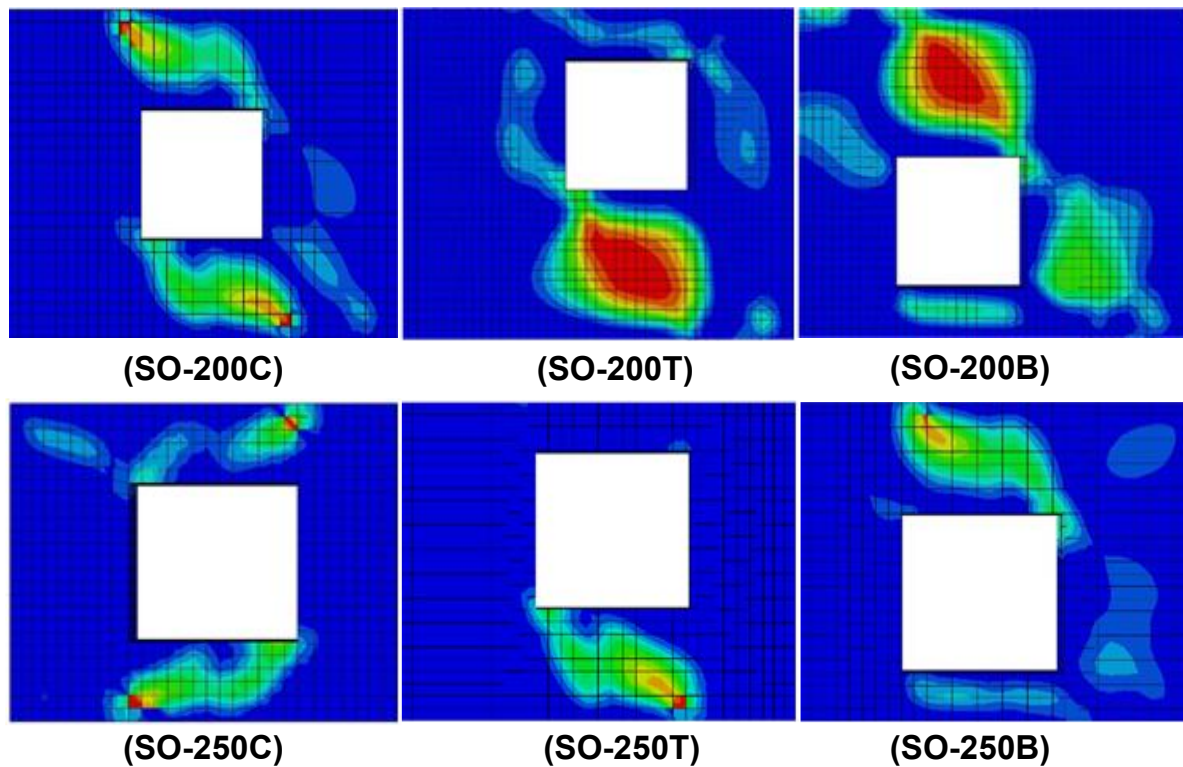


Fig. 7. Comparison of crack propagation patterns in deep beams with square web openings of different sizes and vertical positions.

In specimen SO-200T, the first diagonal crack initiated at the lower corner of the opening and propagated toward the support along the principal load-transfer path. As the applied load increased, another crack developed from the upper corner of the opening toward the loading point, eventually leading to the formation of a shear failure mechanism within the beam web. In contrast, for Specimen SO-200B, where the opening was located in the lower portion of the web, the initial crack propagated from the upper corner of the opening toward the loading region, followed by the development of a second crack extending from the lower corner toward the support. This difference in crack patterns highlights the direct dependence of the failure mechanism on the location of the opening relative to the principal concrete compression strut.

For Specimen SO-200C, in which the opening was positioned at the mid-depth of the web, diagonal cracks initiated almost simultaneously from opposite corners of the opening. This behavior suggests that locating the opening at the center of the web disrupts the principal compression field above and below the opening simultaneously, resulting in the most severe disturbance to the internal stress flow. Furthermore, increasing the opening size from 200 mm to 250 mm led to a noticeable increase in both the width and extent of the diagonal cracks. This observation indicates intensified stress concentration effects and a reduction in the effective concrete area available for force transfer.

3.3. Principal stress distribution

To provide further insight into the mechanisms governing the behavior of the investigated deep beams, the contours of the minimum principal stresses (compressive stresses) and maximum principal stresses (tensile stresses) are presented in Figs. 8 and 9, and Figs. 10 and 11, respectively.

Examination of the minimum principal stress contours indicates that the presence of an opening alters the natural load-transfer path of compressive stresses and causes stress concentrations around the opening. This phenomenon is particularly pronounced in Specimens SO-200C and SO-250C, where the opening is located at the center of the shear span. In these specimens, the opening interrupts the direct path of the principal compression strut extending between the loading point and the support, forcing the compressive stresses to be transferred through the narrow concrete regions above and below the opening. Consequently, stress concentrations increase, leading to a reduction in the load-carrying capacity of the member. In contrast, for Specimen SO-200T, a

substantial portion of the compressive stress field bypasses the opening by passing beneath it and is subsequently transferred to the support. This behavior explains the higher load-carrying capacity observed in this specimen.

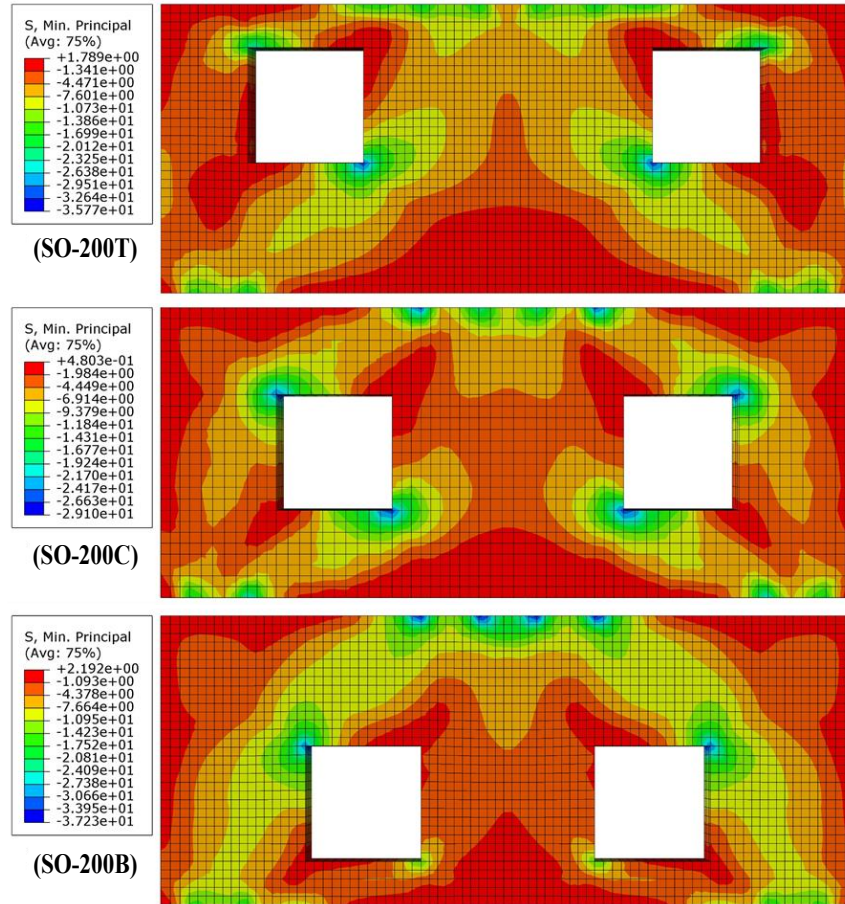


Fig. 8. Comparison of minimum principal stress fields in deep beam specimens with 200×200 mm web openings.

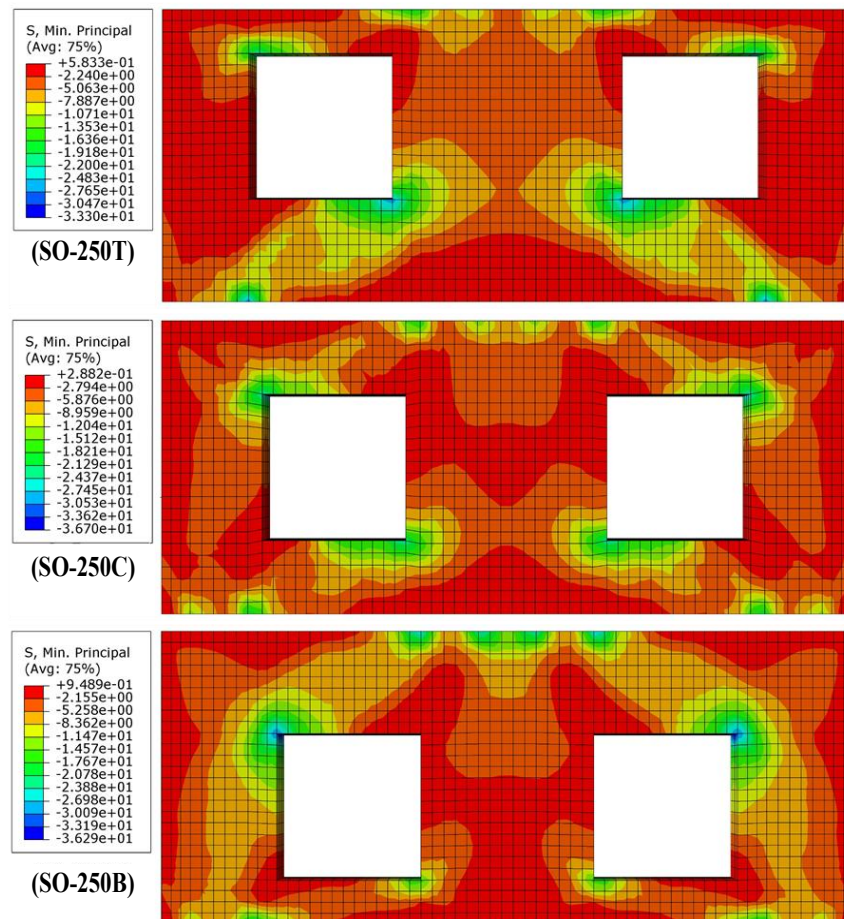


Fig. 9. Comparison of minimum principal stress fields in deep beam specimens with 250×250 mm web openings.

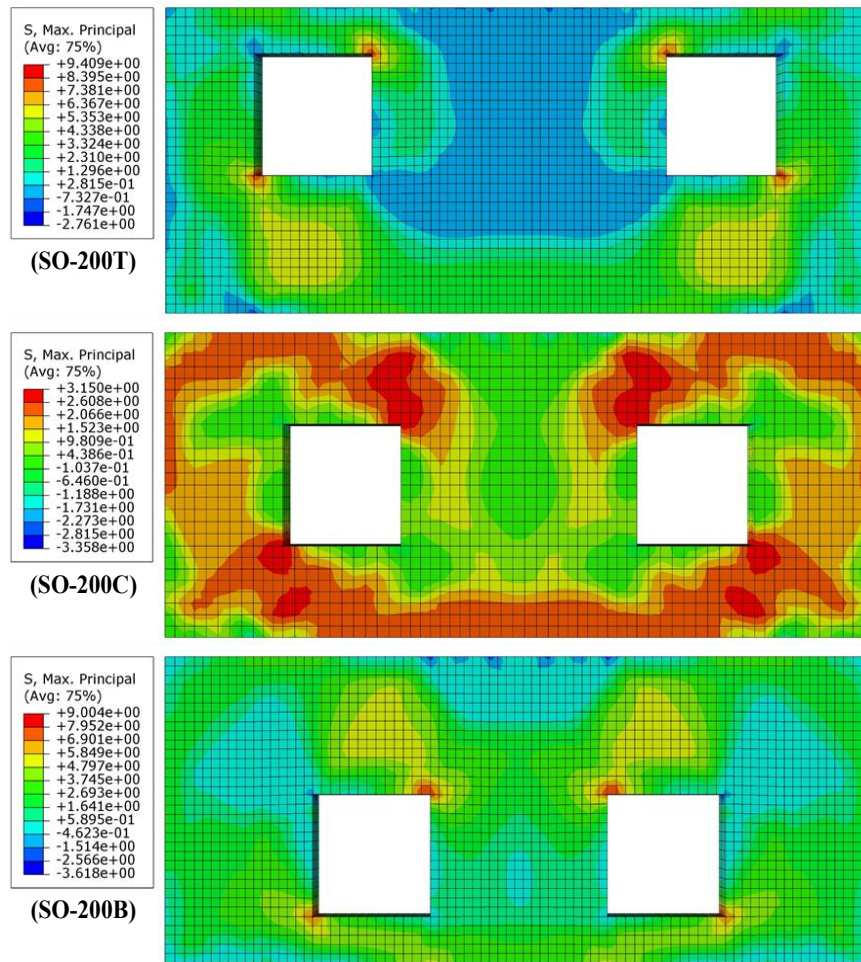


Fig. 10. Comparison of maximum principal stress fields in deep beam specimens with 200×200 mm web openings.

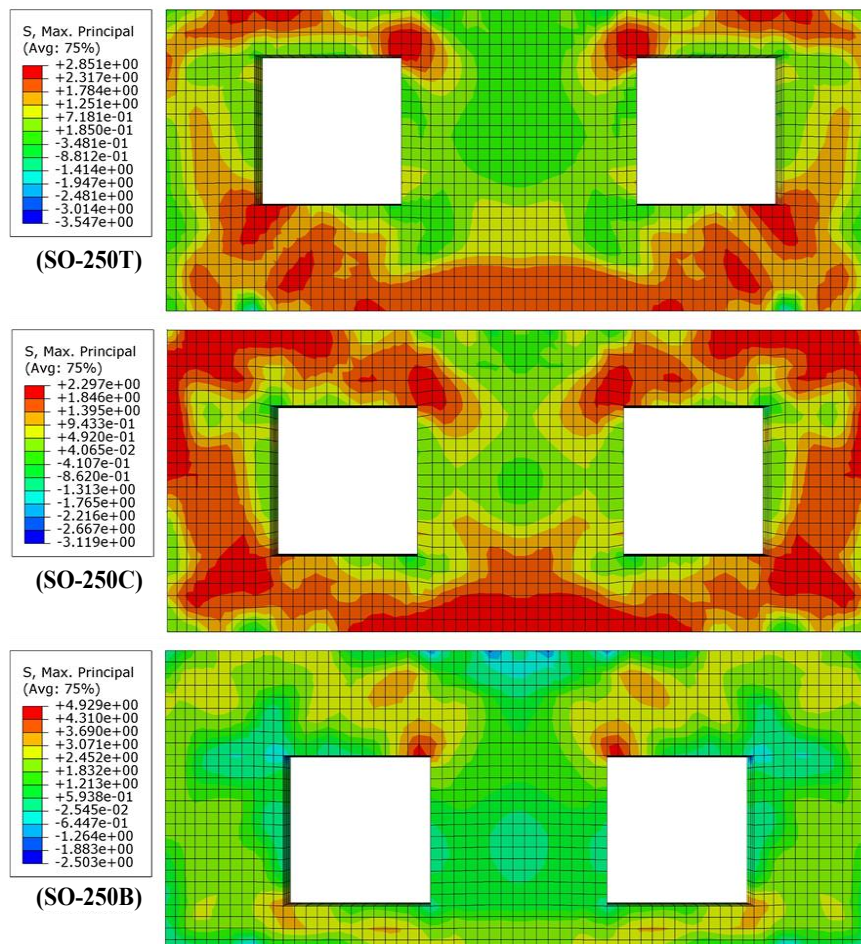


Fig. 11. Comparison of maximum principal stress fields in deep beam specimens with 250×250 mm web openings.

The maximum principal stress contours further reveal that the highest tensile stresses are predominantly concentrated at the corners of the openings, which correspond to the initiation points of the diagonal cracks. Although variations in the size and location of the openings do not significantly alter the overall pattern of stress distribution, they have a substantial influence on the magnitude of local stress concentrations. Within the group containing 200-mm openings, Specimen SO-200T exhibited the highest localized tensile stress, exceeding those of Specimens SO-200C and SO-200B by 198.7% and 4.5%, respectively. Similarly, within the group containing 250-mm openings, Specimen SO-250B experienced the highest tensile stress, with values 114% and 72% greater than those recorded in Specimens SO-250C and SO-250T, respectively. These findings indicate that increasing the opening size becomes particularly detrimental when the opening is located close to the tension zone of the beam, where stress concentrations and crack development become more pronounced.

3.4. Effect of opening location

To isolate the influence of opening location, specimens with identical opening sizes were compared. For both opening sizes (200 mm and 250 mm), centrally located openings produced the greatest deterioration in structural performance because they directly interrupted the principal compression strut connecting the loading point to the support. Conversely, openings located near the top of the web caused the least disturbance to the load-transfer mechanism, while bottom openings exhibited intermediate behavior. This trend was consistently observed in terms of crack propagation, principal stress redistribution, and ultimate load capacity.

3.5. Effect of opening size

To evaluate the influence of opening size independently, specimens with identical opening locations were compared. Increasing the opening size from 200 mm to 250 mm consistently intensified stress concentrations, enlarged the disturbed region, and reduced the effective concrete area available for force transfer. Consequently, larger openings resulted in wider diagonal cracks, greater disruption of the principal compression strut, and lower structural capacity, regardless of the opening location.

3.6. Proposed reinforcement arrangement

The results presented in the preceding sections demonstrated that the presence of a square opening leads to stress concentration at the opening corners, disruption of the principal compression strut, and accelerated propagation of diagonal cracks. Therefore, to improve the structural performance of deep beams without modifying the member geometry or the material properties, a modified web reinforcement arrangement was proposed. The proposed reinforcement arrangement was developed based on the observed stress distribution and crack propagation obtained from the validated finite element analyses. Rather than increasing the total reinforcement ratio uniformly, the modification focused on redistributing the existing web reinforcement toward regions where the principal compression strut was interrupted and stress concentrations were most severe. Additional confinement reinforcement was introduced around the opening corners to improve concrete confinement, restrict diagonal crack propagation, and facilitate stress redistribution within the disturbed region (D-region). Consequently, the proposed detailing aims to restore a more continuous load-transfer mechanism while requiring only a modest increase in the total amount of reinforcing steel. The primary objective of this strategy was to strengthen the load-transfer paths around the opening, enhance concrete confinement within critical regions, and mitigate stress concentration and crack propagation in the vicinity of the opening.

For specimens with centrally located openings (specimens SO-200C and SO-250C), where the greatest disruption of the principal compression strut was observed, two longitudinal web reinforcement bars were relocated to the regions immediately above and below the opening. These bars were intended to act as localized tie elements and redistribute a portion of the forces interrupted by the opening. In addition, supplementary stirrups were provided along both sides and the longitudinal extent of the opening to improve concrete confinement and enhance the stability of the compressive stress fields.

For specimens with openings located in the upper portion of the web (specimens SO-200T and SO-250T), the proximity of the opening to the upper compression reinforcement reduced the need for extensive reinforcement modifications above the opening. In this case, the web reinforcement bars interrupted by the opening were relocated beneath it, and an additional longitudinal reinforcement bar was placed between the opening and the bottom face of the beam. Furthermore, supplementary stirrups were installed around the opening to control stress concentrations and improve force transfer.

For specimens with openings located in the lower portion of the web (specimens SO-200B and SO-250B), the arrangement of the primary bottom flexural reinforcement was maintained unchanged due to the proximity of the opening to the main tension reinforcement. Instead, the web reinforcement bars interrupted by the opening were relocated above it, and an additional longitudinal reinforcement bar was placed between the opening and the top face of the beam within the loading region. Additional stirrups were also provided around the opening to enhance the performance of the discontinuity region (D-region) and to control crack development.

For all specimens, a clear distance of 10 mm, equal to the original concrete cover, was maintained between the reinforcement and the opening edges. This measure ensured constructability while preventing excessive reinforcement congestion around the opening. It should be noted that implementation of the proposed reinforcement scheme resulted in only a limited increase in the amount of reinforcing steel, ranging from approximately 4% to 8.4%. Therefore, the proposed solution can be considered both practical and economically viable. Fig. 12 presents the detailed reinforcement layouts adopted for the different specimens. The specimens incorporating the modified reinforcement arrangement are designated by the suffix “-M”; for example, SO-200C-M denotes the modified version of specimen SO-200C.

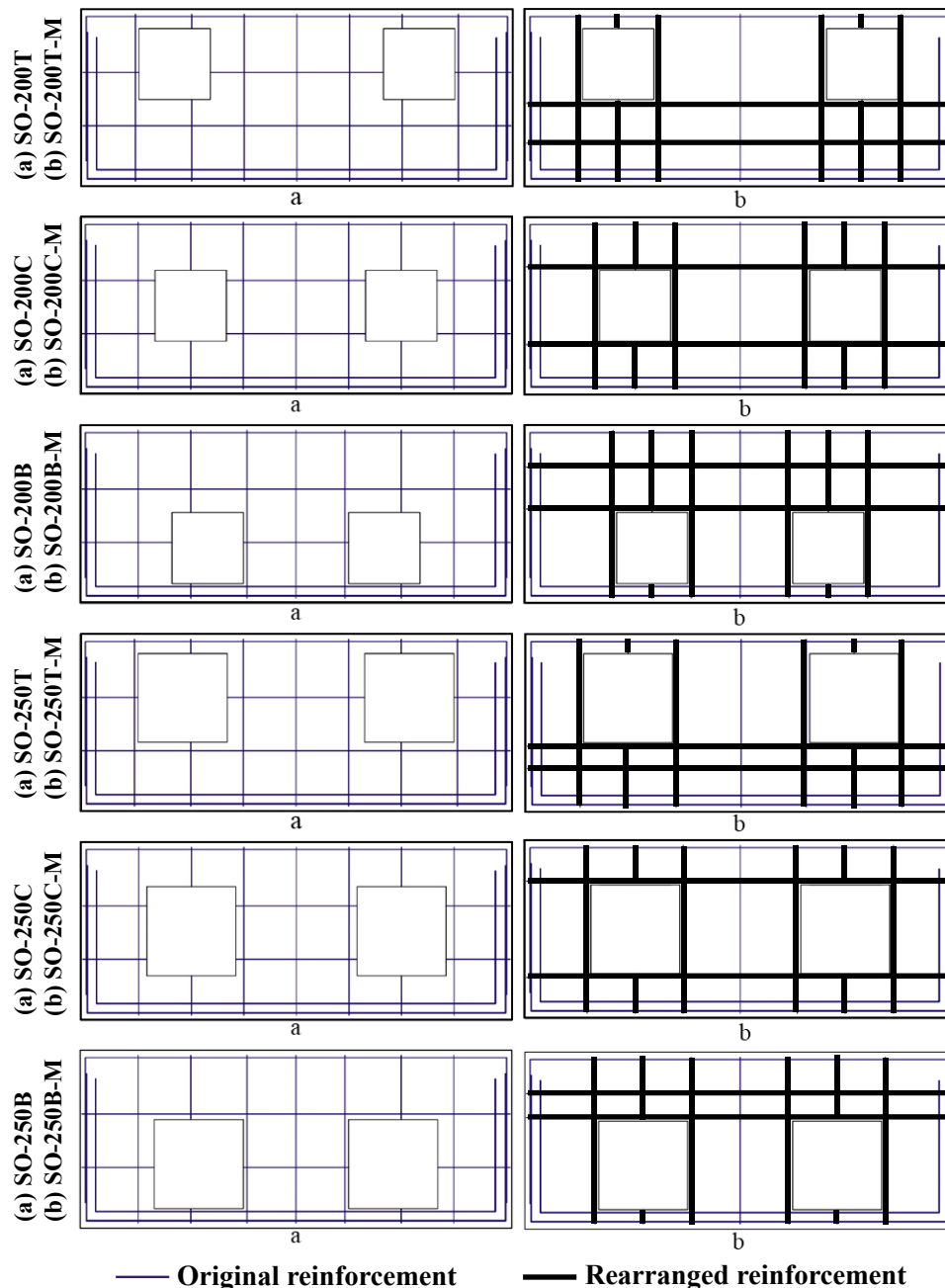


Fig. 12. Comparison of: (a) the original reinforcement layout; and (b) the modified reinforcement layout proposed for deep beam specimens with square web openings.

3.6.1. Effect on load-displacement response

To evaluate the effectiveness of the proposed reinforcement arrangement, nonlinear finite element analyses were repeated for all modified specimens, and the resulting midspan load-displacement curves were compared with those of the original specimens. Fig. 13 presents a comparison of the load-displacement responses before and after the reinforcement modification.

Examination of these curves indicates that the proposed reinforcement arrangement had a significant influence on the overall behavior of the deep beams with web openings. Although the initial portions of the curves, corresponding to the elastic response of the members, exhibit only minor differences in most specimens, the effect of the reinforcement modification becomes clearly evident after crack initiation. In all specimens, the redistribution of web reinforcement resulted in an increase in the cracking load, improved stability of the load-carrying mechanism, and enhanced ultimate load-carrying capacity.

For the specimens with 200-mm openings, the ultimate load-carrying capacity increased by approximately 3% to 9%. Moreover, the load-displacement curves indicate that, following the formation of the initial cracks, the reduction in structural stiffness occurred at a slower rate in the modified specimens. Consequently, these specimens were capable of sustaining higher loads at comparable displacement levels. This behavior suggests that the reinforcement relocated around the openings effectively contributed to partially restoring the load-transfer mechanism interrupted by the presence of the openings.

A similar trend was observed for the specimens with 250-mm openings. However, the influence of the reinforcement modification was more pronounced in this group, resulting in increases in ultimate load-carrying capacity ranging from 5% to 10%. This finding indicates that as the opening size increases and the disturbance to the internal stress field becomes more severe, the

contribution of the additional reinforcement to force redistribution and control of the discontinuity region becomes increasingly important. In other words, the greater the reduction in the effective web area caused by the opening, the more effective reinforcement-detailing strategies become in mitigating the associated loss of structural performance.

As shown in Fig. 13 further reveals that the specimens with centrally located openings experienced the greatest improvement. This behavior can be attributed to the fact that, in these specimens, the opening causes the most severe interruption of the principal compression strut. Consequently, the additional reinforcement placed around the opening is able to play a more effective role in establishing alternative load-transfer paths. In contrast, the specimens with upper openings, which initially caused the least disturbance to the principal compression strut, exhibited relatively smaller improvements following the reinforcement modification.

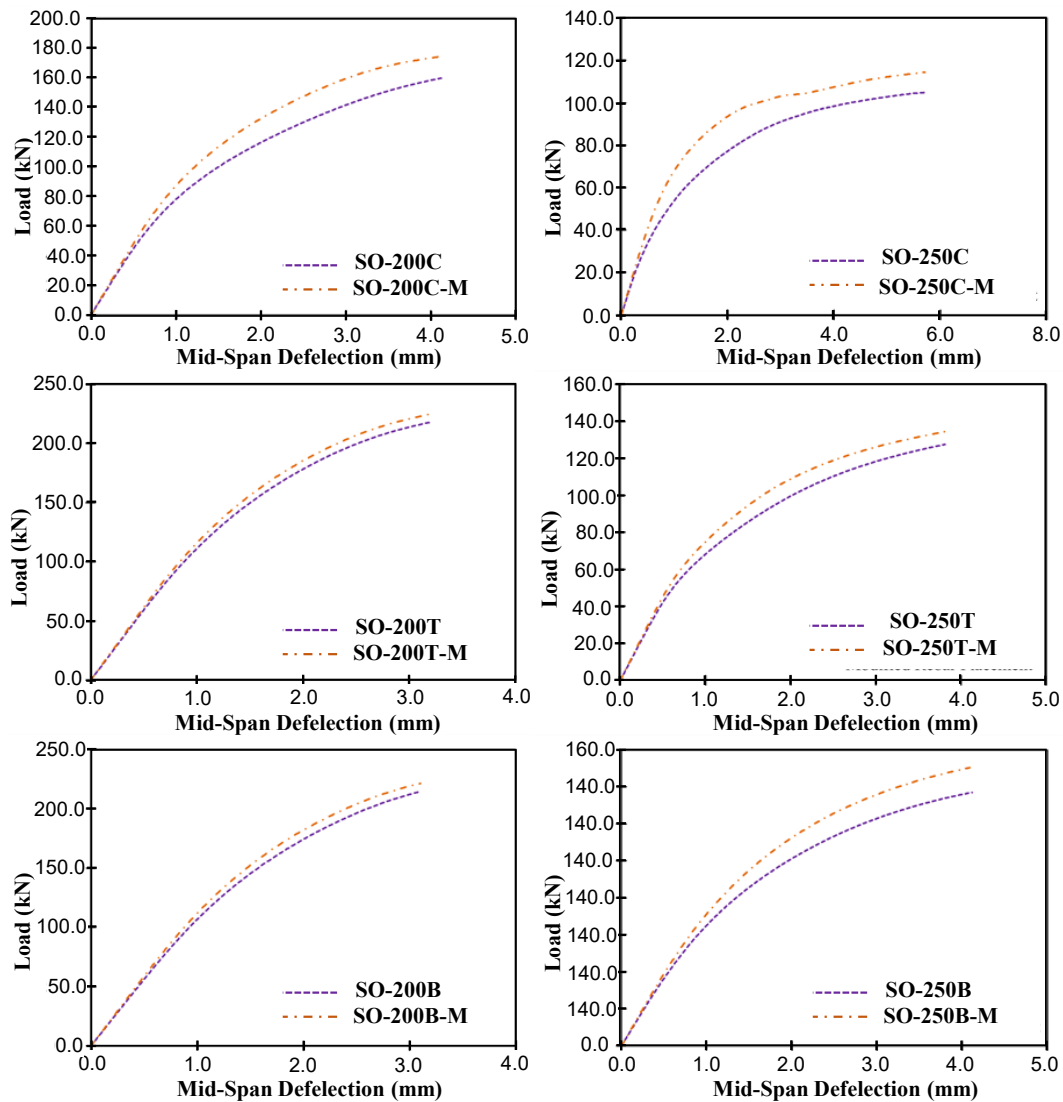


Fig. 13. Comparison of midspan load-displacement behavior of deep beam specimens before and after reinforcement layout modification.

Overall, the load-displacement responses demonstrate that the strategic concentration of web reinforcement in the vicinity of the openings not only enhances the ultimate load-carrying capacity but also improves the nonlinear behavior and structural stability of deep beams by delaying damage initiation and reducing the rate of stiffness degradation after cracking. These findings confirm that reinforcement rearrangement can serve as a practical and cost-effective strengthening strategy for compensating a portion of the performance degradation associated with the introduction of openings in reinforced concrete deep beams.

3.6.2. Effect on crack development and cracking load

The crack patterns and contours of maximum tensile plastic strains for the modified specimens are presented in Fig. 14. Comparison of the results before and after reinforcement rearrangement indicates that relocating the web reinforcement and providing additional stirrups around the openings effectively improved the stress transfer mechanism within the disturbed regions (D-regions) and reduced local stress concentrations at critical locations.

The beneficial effect of the proposed reinforcement arrangement was particularly evident in specimens SO-250C-M and SO-250B-M, which contained the larger square openings located at the central and lower levels of the web, respectively. In these specimens, the additional reinforcement surrounding the opening enhanced concrete confinement and promoted a more uniform distribution of internal forces around the discontinuity. As a result, the tensile plastic strains previously concentrated at the opening corners were significantly reduced.

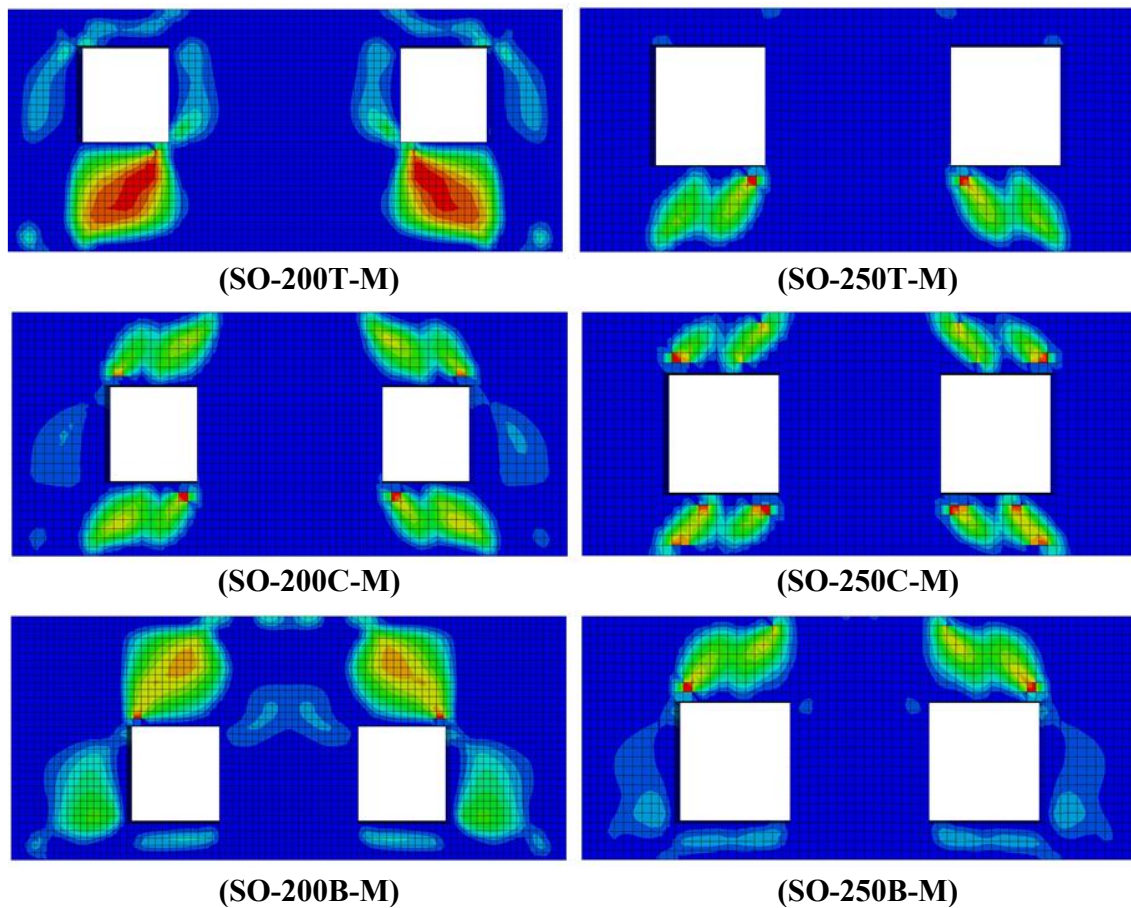


Fig. 14. Crack patterns of reinforced concrete deep beams with square web openings after reinforcement rearrangement.

A notable observation is that the stirrups placed adjacent to the opening restricted the outward propagation of tensile plastic strain zones originating from the upper corners of the opening. Consequently, crack development became more localized and controlled, indicating an improvement in the structural behavior of the discontinuity region. Although the overall crack trajectories remained generally similar to those observed in the original specimens, the intensity and extent of critical cracking were noticeably reduced after reinforcement modification.

In addition to crack control, the proposed reinforcement arrangement improved the nonlinear deformation capacity of the beams. The maximum tensile plastic strain increased by 16.4%, 2.67%, and 48.8% in specimens SO-200C-M, SO-200T-M, and SO-200B-M, respectively, compared with their corresponding reference specimens. These increases indicate a greater ability of the modified specimens to redistribute stresses and sustain inelastic deformations prior to failure. The increase in tensile plastic strain capacity suggests that the modified specimens were able to sustain larger inelastic deformations prior to failure.

Overall, the results demonstrate that although reinforcement rearrangement does not fundamentally alter the governing cracking mechanism, it effectively mitigates stress concentration around the opening, improves concrete confinement, and enhances the nonlinear response of deep beams. These findings highlight the importance of reinforcement detailing in the vicinity of web openings and confirm that an appropriate redistribution of web reinforcement can significantly improve the cracking performance and overall structural behavior of reinforced concrete deep beams with square openings.

A quantitative assessment of the effectiveness of the proposed reinforcement arrangement was performed by comparing the results of the modified specimens presented in Table 2 with those of the original specimens. The comparison indicates that the reinforcement modification successfully delayed the initiation of the first diagonal shear crack in all specimens. Depending on the opening size and location, the cracking load increased by 4.5% to 44%, demonstrating the effectiveness of the proposed detailing in improving the cracking resistance of deep beams with web openings.

The smallest improvement was observed in specimens with openings located near the upper part of the web (specimens SO-200T-M and SO-250T-M). In these specimens, the openings caused relatively limited interference with the main compression strut, and therefore the potential for further improvement was inherently restricted. In contrast, the largest enhancement occurred in specimens with centrally located openings (specimens SO-200C-M and SO-250C-M), where the opening interrupted the primary load-transfer path most severely. In such cases, the relocated reinforcement effectively restored part of the disrupted force-transfer mechanism, resulting in a substantial increase in cracking load.

The results further show that the (P_{cr} / P_u) ratio increased in all modified specimens, with improvements ranging from 1% to 33%. Since this ratio reflects the relative delay in the onset of nonlinear behavior, the observed increase indicates that the proposed reinforcement arrangement enhanced the structural stability of the beams and postponed crack propagation under increasing load levels.

The influence of opening size was also found to be significant. As the opening dimension increased from 200 mm to 250 mm, the effectiveness of the reinforcement modification became more pronounced. For example, the increase in cracking load achieved in the specimens with 250-mm openings was substantially greater than that obtained for the corresponding specimens with 200-mm openings. This trend suggests that the structural benefits of reinforcement redistribution become increasingly important as the severity of the geometric discontinuity increases. Therefore, appropriate reinforcement detailing around large web openings can play a critical role in controlling stress concentration and improving force redistribution within disturbed regions (D-regions).

Table 2. Comparison of cracking and ultimate loads of deep beams with openings before and after reinforcement layout modification.

Specimen	Original reinforcement layout			Modified reinforcement layout			Increase in cracking load (%)
	Cracking load (kN)	Ultimate load (kN)	P_{cr} / P_u	Cracking load (kN)	Ultimate load (kN)	P_{cr} / P_u	
SO-200C	43.16	159.64	0.27	62.31	174.21	0.36	44.38
SO-200T	75.55	208.08	0.35	78.99	224.83	0.35	4.55
SO-200B	81.62	214.53	0.38	91.95	221.39	0.41	12.65
SO-250C	41.93	104.94	0.40	52.02	114.44	0.46	24.06
SO-250T	42.86	127.54	0.34	48.92	134.51	0.36	14.13
SO-250B	36.16	136.76	0.26	47.60	150.37	0.32	31.6

3.6.3. Effect on minimum principal stress distribution

To evaluate the influence of reinforcement rearrangement on the load-transfer mechanism and the performance of the discontinuity region surrounding the opening, the contours of the minimum principal stresses (compressive stresses) after the reinforcement modifications are presented in Figs. 15 and 16 for specimens with 200-mm and 250-mm openings, respectively. Comparison of these contours with those obtained from the original specimens indicates that relocating a portion of the web reinforcement around the opening and increasing the reinforcement density within critical regions resulted in a more uniform distribution of compressive stresses and a reduction in stress concentrations at the opening corners. Consequently, the load-transfer paths around the opening became more organized, and the disturbance introduced into the compressive stress field was partially mitigated.

The quantitative results further support these observations. For the specimens with 200-mm openings, the magnitude of the minimum principal stresses in the vicinity of the opening increased by 57.6%, 14.5%, and 60.2% for specimens SO-200C-M, SO-200T-M, and SO-200B-M, respectively. Similarly, for the specimens with 250-mm openings, the corresponding increases for specimens SO-250C-M, SO-250T-M, and SO-250B-M were 40.3%, 37.5%, and 29.5%, respectively. These results indicate that the effectiveness of the reinforcement modification was more pronounced in specimens with openings located in the central or lower regions of the web. In these cases, the openings caused more severe interference with the principal compression strut, allowing the reinforcement rearrangement to play a more significant role in stress redistribution.

A comparison between the two opening groups further reveals that, although the reinforcement modification improved the distribution of compressive stresses in all specimens, the magnitude of this improvement decreased in some cases as the opening size increased from 200 mm to 250 mm. This behavior suggests that as the geometric discontinuity becomes larger, the disturbance to the load-transfer mechanism intensifies, thereby limiting the ability of the additional reinforcement to redistribute stresses effectively.

Overall, the results presented in Figs. 15 and 16 demonstrate that the proposed reinforcement arrangement successfully improved the structural behavior of deep beams with square openings by reducing compressive stress concentrations around the openings and enhancing the performance of the concrete compression struts. These findings highlight the importance of reinforcement detailing within discontinuity regions (D-regions) and emphasize its significant role in improving the nonlinear response of reinforced concrete deep beams.

3.6.4. Effect on maximum principal stress distribution

To assess the influence of reinforcement rearrangement on the tensile stresses governing crack initiation and propagation in concrete, the contours of the maximum principal stresses after the reinforcement modifications are presented in Figs. 17 and 18 for specimens with 200-mm and 250-mm openings, respectively. A comparison of the results before and after the reinforcement modification indicates that, although the reinforcement rearrangement led to a more balanced stress distribution around the openings, the overall pattern of maximum principal stress distribution remained largely unchanged. The critical tensile stress regions continued to be concentrated primarily at the corners of the openings.

Examination of the quantitative results for the maximum principal stresses reveals that, following the reinforcement modification, these stresses increased by 4.9%, 7.5%, and 10.3% for specimens SO-200C-M, SO-200T-M, and SO-200B-M, respectively. Similarly, increases of 0.5% and 10.85% were observed for specimens SO-250T-M and SO-250B-M, whereas Specimen SO-250C-M exhibited a very slight reduction of 0.56% in the maximum principal stress. These findings indicate that, unlike the compressive stress field, reinforcement rearrangement does not directly reduce the peak tensile stress levels, and stress concentration at the opening corners remains a dominant characteristic of the structural behavior in this region.

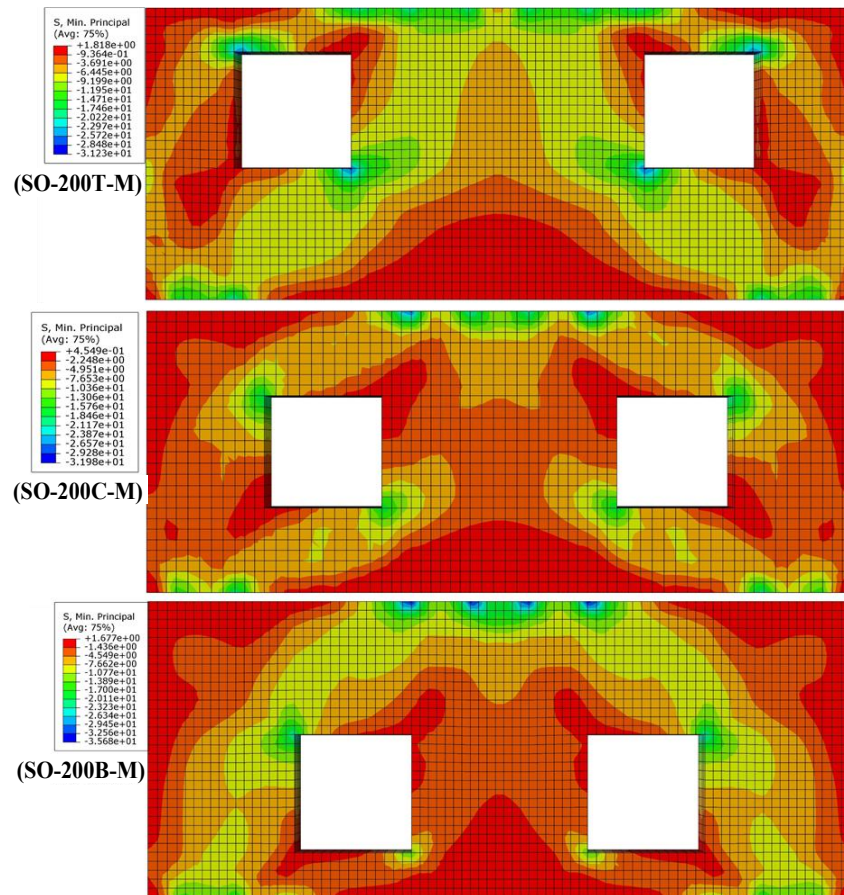


Fig. 15. Comparison of minimum principal stress fields in reinforced deep beam specimens with modified reinforcement layouts and 200×200 mm web openings.

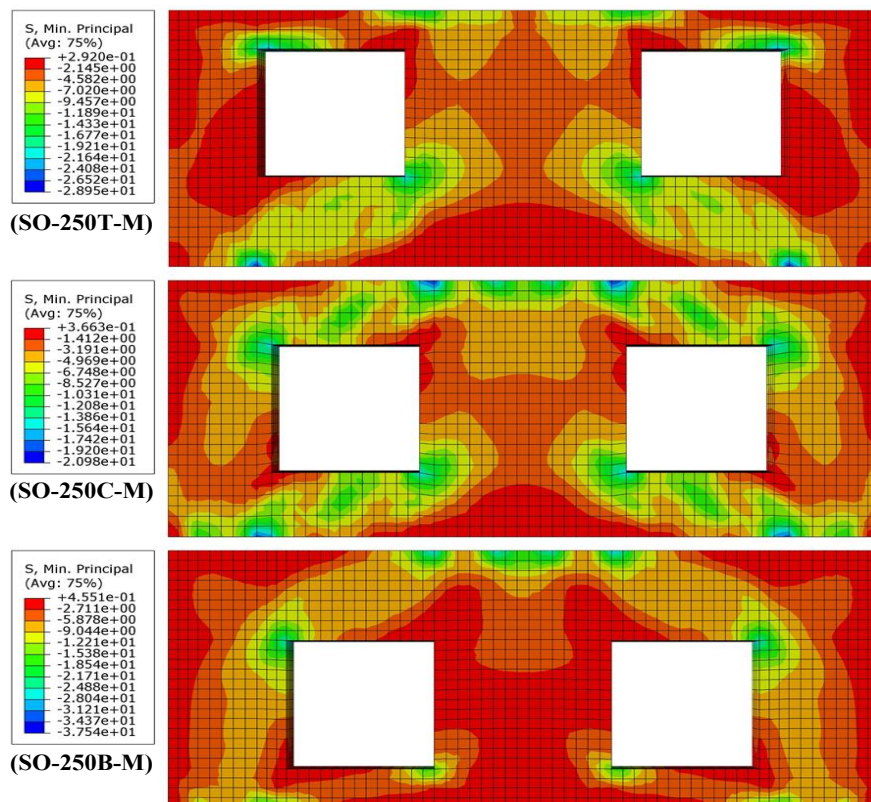


Fig. 16. Comparison of minimum principal stress fields in reinforced deep beam specimens with modified reinforcement layouts and 250×250 mm web openings.

This behavior can be attributed to the inherent nature of tensile stress concentrations within discontinuity regions. Even with modifications to the reinforcement layout and improvements in concrete confinement, the locations of critical tensile stresses remain largely unchanged. Nevertheless, a comparison of the contours presented in Figs. 17 and 18 demonstrates that the reinforcement rearrangement resulted in a more uniform stress distribution around the openings and facilitated the transfer of a portion of the localized stresses to adjacent regions. This stress redistribution, together with the observed increases in cracking load and ultimate

load-carrying capacity, indicates that although the effect of reinforcement modification on the peak tensile stress values is limited, it plays a significant role in improving the overall performance of the discontinuity region and enhancing the structural behavior of reinforced concrete deep beams with square openings.

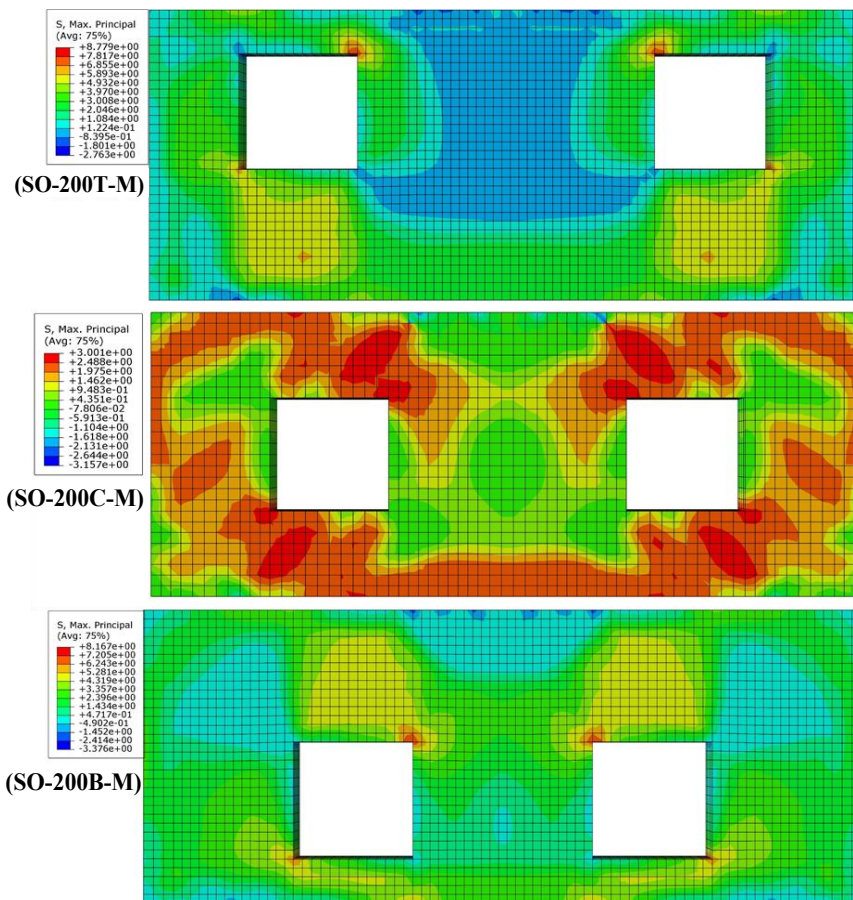


Fig. 17. Comparison of maximum principal stress fields in reinforced deep beam specimens with modified reinforcement layouts and 200×200 mm web openings.

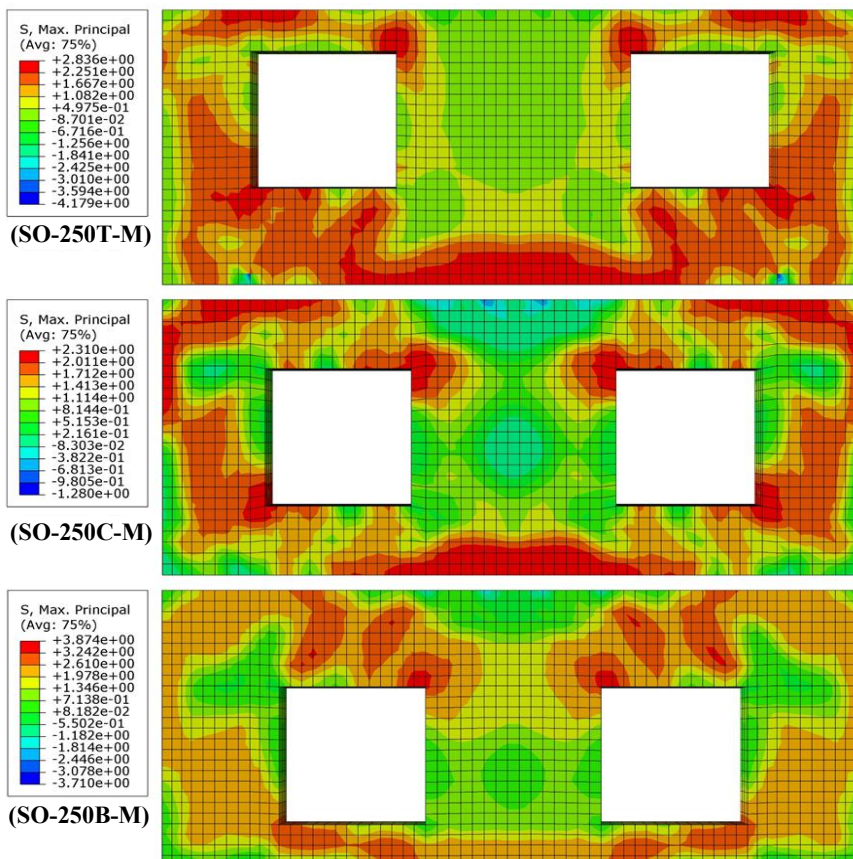


Fig. 18. Comparison of maximum principal stress fields in reinforced deep beam specimens with modified reinforcement layouts and 250×250 mm web openings.

For ease of comparison, the principal quantitative results obtained from the parametric study are summarized in Table 3. The table highlights the influence of opening size, opening location, and reinforcement rearrangement on the ultimate load capacity, cracking load, stress redistribution, and structural performance of the investigated deep beams.

Table 3. Summary of the influence of opening configuration and reinforcement detailing on the structural performance of reinforced concrete deep beams.

Specimen	Ultimate load (kN)	Cracking load (kN)	P_{cr} / P_u	Principal stress redistribution	Overall structural performance
SO200-C	159.64	47	0.29	Severe disruption of the principal compression strut	Most adversely affected by opening
SO200-T	218.08	82.28	0.37	Compression stresses largely bypass the opening	Least affected by opening
SO200-B	214.53	84.13	0.39	Partial disruption of the compression strut	Moderately affected by opening
SO250-C	104.53	39.26	0.37	Highly disturbed compression stress field	Most severely affected by opening size
SO250-T	127.54	42.86	0.34	Moderate stress redistribution	Moderately affected by opening size
SO250-B	136.76	53.98	0.39	Significant stress redistribution	Significantly affected by opening size
SO200-C-M	174.21	62.31	0.36	Improved stress redistribution	Structural performance substantially recovered
SO200-T-M	224.83	78.99	0.35	Slightly improved stress distribution	Structural performance slightly improved
SO200-B-M	221.39	91.95	0.41	Improved stress redistribution	Structural performance moderately improved
SO250-C-M	114.44	52.03	0.45	Significantly improved stress redistribution	Structural performance markedly improved
SO250-T-M	134.51	48.92	0.36	Moderately improved stress redistribution	Structural performance moderately improved
SO250-B-M	150.37	47.60	0.31	Enhanced stress redistribution around the opening	Structural performance significantly improved

4. Engineering implications

The findings of this study demonstrate that both the size and vertical location of web openings have a significant influence on the structural behavior of reinforced concrete deep beams. Within the investigated configurations, openings located at the mid-depth of the web caused the greatest disruption to the principal load-transfer path, resulting in reduced load-carrying capacity, increased stress concentrations, and more severe crack propagation. Furthermore, increasing the opening size intensifies these adverse effects and increases the sensitivity of the member to geometric discontinuities.

Conversely, the strategic redistribution of web reinforcement around the opening can significantly improve the structural performance of deep beams without requiring a substantial increase in the amount of reinforcing steel. The observed increases in cracking load, load-carrying capacity, deformation capacity, and the more uniform distribution of compressive stresses indicate that reinforcement detailing within discontinuity regions (D-regions) plays a critical role in controlling the behavior of deep beams containing web openings. Therefore, the results suggest that, in addition to the size and location of the opening, reinforcement detailing around the opening should be considered as an important design parameter for improving force transfer and structural performance. The proposed reinforcement arrangement provides a practical and economical approach for mitigating the adverse effects of square web openings in reinforced concrete deep beams without requiring significant modifications to member geometry. It should be noted that the proposed reinforcement detailing is intended as a practical detailing concept for deep beams with structural characteristics similar to those investigated in the present study. Further studies covering a wider range of beam geometries and loading conditions are recommended before generalizing the proposed detailing to broader design applications.

5. Conclusions

In this study, the structural behavior of reinforced concrete deep beams containing square web openings was investigated through nonlinear finite element analysis using the concrete damaged plasticity (CDP) model in ABAQUS [25]. The validated finite element model accurately reproduced the experimental load-displacement response, crack patterns, and ultimate load capacity, providing a reliable basis for the subsequent parametric investigation of deep beams with square web openings. The effects of opening size, opening location, and reinforcement rearrangement around the openings on crack propagation, stress distribution, and load-carrying capacity were examined. Based on the obtained results, the following conclusions can be drawn:

- The location of the opening was found to be one of the most influential parameters affecting the structural performance of deep beams. Openings located at the center of the shear span caused the greatest disruption to the main compression strut, resulting in the most severe disturbance of the internal force-transfer mechanism and the largest reduction in load-carrying capacity among the investigated specimens.
- Crack initiation and propagation were strongly dependent on both the size and vertical position of the opening. Due to the geometric discontinuity introduced by square openings, stress concentration occurred primarily at the opening corners, where diagonal shear cracks were initiated and subsequently propagated toward the loading and support regions.
- Increasing the opening size from 200 mm to 250 mm intensified stress concentration, increased the extent of diagonal cracking, and aggravated the disturbance of the compression stress field.

- Analysis of the principal stress fields demonstrated that the presence of square openings significantly altered the natural stress flow within the deep beam. Openings positioned along the main load-transfer path forced the compressive stresses to bypass the opening through narrow concrete zones, leading to localized stress concentrations and reduced load-carrying capacity.
- The proposed reinforcement rearrangement strategy, consisting of relocating web reinforcement around the opening and providing additional confinement reinforcement adjacent to critical regions, proved effective in enhancing the structural performance of the beams while requiring only a modest increase in steel consumption.
- The modified reinforcement layouts increased the ultimate load capacity by approximately 3-10% and delayed the initiation of diagonal cracking by 4.5-44%, depending on the opening size and location. Furthermore, the ratio of cracking load to ultimate load (P_{cr} / P_u) increased in all investigated specimens, indicating a delay in crack initiation and the onset of nonlinear behavior.
- Reinforcement modification improved the distribution of compressive stresses around the openings and reduced stress concentration in critical regions. Although its influence on the magnitude of peak tensile stresses was relatively limited, it significantly enhanced crack control, deformation capacity, and overall structural performance
- The effectiveness of the proposed reinforcement strategy became more pronounced as the opening size increased. This finding highlights the critical role of reinforcement detailing in discontinuity regions (D-regions), particularly in deep beams with large web openings where the disturbance of force flow is more severe.

Statements & declarations

Author contributions

Amir Keshavarz: Investigation, Formal analysis, Writing - Original Draft.

Ebrahim Fadaei: Methodology, Writing - Review & Editing.

Mohammad Ali Dastan Diznab: Project administration, Resources, Methodology, Conceptualization.

Seyed Hamid Hashemi: Methodology, Conceptualization.

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Data availability

The datasets analyzed in the current study are available from the corresponding author on reasonable request.

Declarations

The authors declare no conflict of interest.

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Size-Dependent Constitutive Model for Normal- and High-Strength Unconfined Concrete

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ABSTRACT

This study presents a unified stress-strain model for unconfined concrete under axial compression that accounts for key size-dependent effects and varying material strength levels. The proposed formulation is designed to overcome limitations in existing constitutive models, particularly their restricted applicability across different specimen geometries, strength classes, and their limited ability to consistently represent both pre-peak and post-peak behavior within a single coherent framework. The model adopts a two-stage representation of the compressive response, allowing independent yet physically consistent characterization of the nonlinear ascending branch and the strain-softening post-peak regime. The formulation explicitly incorporates the influence of concrete strength and specimen geometry, enabling the model to capture transitions from more ductile behavior in lower-strength concrete to increasingly brittle responses in higher-strength materials. The proposed approach is calibrated and validated using an extensive experimental database covering a wide range of concrete strengths, specimen dimensions, and aspect ratios compiled from multiple independent research programs. Comparisons with experimental stress-strain curves demonstrate that the model reliably reproduces both the peak response and the subsequent softening behavior across diverse testing conditions, including cases with pronounced geometric and material variability. The results confirm that the proposed formulation provides a simplified but robust representation of the compressive behavior of unconfined concrete, making it suitable for nonlinear structural analysis and performance-based engineering applications without the need for case-specific recalibration.

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1. Introduction

Concrete remains the most widely used construction material in civil infrastructure due to its high compressive strength, durability, versatility, and economic efficiency. The accurate characterization of its mechanical response under compression is fundamental for the analysis, design, assessment, and numerical simulation of reinforced concrete structures. In particular, the axial stress-strain relationship of unconfined concrete constitutes a key constitutive law employed in nonlinear finite element analyses, performance-based structural design, and the evaluation of structural safety under extreme loading conditions. Since the pioneering work of Hognestad [1], numerous analytical models have been proposed to represent the compressive behavior of concrete, including those developed by Kent and Park [2], Popovics [3], Mander et al. [4], Collins et al. [5], Légeron and Paultre [6], Karthik Madhu and Mander John [7], Lim and Ozbakkaloglu [8], Yang et al. [9], Shayanfar et al. [10]. Although these models have significantly advanced the understanding of concrete constitutive behavior, the increasing availability of experimental evidence

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covering broader strength ranges and specimen dimensions has highlighted the need for more versatile and physically representative formulations. In particular, the influence of size effect on compressive behavior, together with the transition from normal- to high-strength concrete, requires constitutive models capable of providing reliable predictions across a wide spectrum of material and geometric conditions.

The development of stress-strain models for unconfined concrete has evolved substantially over the past several decades; however, several limitations remain that restrict the accuracy, robustness, and practical applicability of existing formulations. A primary concern is the mathematical complexity adopted by some constitutive models. To improve curve-fitting performance, several researchers have introduced formulations involving numerous empirical coefficients, higher-order functions, or iterative calibration procedures (e.g., Karthik Madhu and Mander John [7]). Although these approaches may provide satisfactory agreement with specific experimental datasets, they often reduce transparency regarding the physical meaning of model parameters and complicate their implementation in design-oriented calculations and nonlinear numerical analyses. From an engineering perspective, a constitutive model should not only reproduce experimental observations with sufficient accuracy but should also maintain computational efficiency, numerical stability, and ease of calibration using readily available material properties. Another important limitation relates to the use of a single analytical function to represent the complete compressive response of concrete, including both the pre-peak hardening and post-peak softening regions. The mechanical mechanisms governing these two stages are fundamentally different. The ascending branch is primarily controlled by microcrack initiation, progressive crack accumulation, and material compaction, whereas the descending branch is dominated by crack localization, fracture propagation, aggregate interlock degradation, and progressive loss of load-carrying capacity (Tariq et al. [11], Shayanfar et al. [12]). Consequently, enforcing a single continuous mathematical expression over the entire stress-strain domain requires the simultaneous satisfaction of multiple boundary conditions associated with both regions. Such constraints often reduce the flexibility of the model to independently capture the curvature of the ascending branch and the rate of post-peak deterioration. This limitation becomes particularly evident in the softening regime, where small variations in material properties or specimen geometry can produce significant changes in the shape of the stress-strain response (Shayanfar et al. [13, 14]). As a result, models employing a unified function frequently exhibit reduced accuracy in predicting strain-softening behavior, which is critical for evaluating ductility, energy dissipation capacity, and failure progression in concrete structures. The applicability of many existing models is further constrained by their calibration range. A considerable proportion of constitutive formulations were developed using experimental datasets dominated by normal-strength concrete, typically within a relatively narrow strength interval. However, the mechanical response of concrete changes significantly with increasing compressive strength (e.g., Yang et al. [9]). High-strength concrete generally exhibits a denser microstructure, reduced internal porosity, and improved matrix-aggregate bonding, resulting in higher peak stresses but also increased brittleness and steeper post-peak degradation (Eid et al. [15], Shayanfar et al. [16–19]). Consequently, parameters calibrated for normal-strength concrete cannot necessarily reproduce the behavior of high-strength concrete with sufficient accuracy. Similarly, constitutive parameters derived from high-strength concrete datasets may overestimate brittleness when applied to lower-strength materials. This dependence on strength class limits the generality of many existing formulations and necessitates the development of unified models capable of accurately representing a broad spectrum of concrete strengths without requiring separate calibration procedures. A further limitation concerns the insufficient consideration of size effect in many stress-strain relationships. Concrete is a quasi-brittle material whose mechanical response is strongly influenced by specimen dimensions due to the heterogeneous nature of its internal structure and the localization of fracture processes. Experimental investigations (e.g., Gholampour et al. [20]) have consistently demonstrated that increasing specimen size leads to reductions in nominal compressive strength and alterations in deformation capacity. These effects originate from the statistical distribution of flaws, the development of localized fracture zones, and the non-uniform dissipation of fracture energy within the material (Shayanfar and Barros Joaquim [21]). Larger specimens tend to exhibit more pronounced strain localization and a more brittle post-peak response than smaller specimens (Shayanfar et al. [22], Watanabe et al. [23]). Nevertheless, a substantial number of constitutive models have been developed primarily from tests performed on standard cylinders or relatively small laboratory specimens, implicitly assuming geometric independence of the stress-strain response. Such an assumption can result in significant prediction errors when the models are applied to structural members with dimensions substantially different from those used during calibration (Shayanfar et al. [24, 25]). Although some researchers have attempted to incorporate size-effect phenomena into constitutive formulations, these approaches frequently involve complex analytical procedures, multiple correction factors, or intermediate calculations that reduce their practicality for engineering applications. In many cases, the implementation of size-dependent formulations requires the determination of parameters that are not routinely measured in experimental programs or available in design specifications (Paultre et al. [26], Sadeghpour Haji et al. [27]). Consequently, despite their improved theoretical rigor, such models may be difficult to integrate into structural analysis software or everyday engineering practice. The challenge therefore lies in developing a formulation that captures the essential physics of size-dependent behavior while maintaining a level of simplicity suitable for widespread application. Finally, the predictive reliability of many existing models is influenced by the scope and quality of the experimental databases used during their development. Several constitutive relationships were calibrated using relatively limited datasets covering a restricted range of concrete strengths, specimen dimensions, and geometric configurations. Such datasets may not adequately capture the inherent variability associated with concrete behavior, including differences in aggregate characteristics, curing conditions, specimen geometry, loading procedures, and material composition. As a consequence, the resulting formulations may exhibit satisfactory performance within the calibration domain but reduced predictive capability when extrapolated to conditions outside that range. The increasing availability of extensive experimental databases now provides an opportunity to establish statistically robust constitutive relationships whose parameters reflect a broader spectrum of material and geometric characteristics. Collectively, these limitations highlight the need for a new generation of stress-strain models that integrate physical realism, size-effect sensitivity, computational simplicity, and broad applicability within a unified constitutive framework.

To address these challenges, the present study proposes a new size-dependent stress-strain model for unconfined concrete subjected to axial compression. The proposed methodology adopts a two-branch constitutive representation consisting of a polynomial-based ascending branch and a logistic-type descending branch, enabling independent calibration of the pre-peak and post-peak responses and thus improving the accuracy of softening behavior prediction. The model explicitly incorporates the effects of concrete strength, specimen dimensions, and aspect ratio through data-driven formulations calibrated using an extensive experimental database covering a wide range of concrete classes and geometrical configurations. In addition, size-effect formulations are incorporated directly into the determination of peak stress and corresponding strain, ensuring consistency with experimentally observed scaling phenomena. The resulting model aims to provide a unified, computationally efficient, and statistically robust constitutive relationship capable of accurately reproducing the stress-strain response of unconfined concrete, from initial loading to post-peak degradation, while maintaining applicability across different specimen sizes and strength levels.

2. Estimation of peak stage parameters

To assess the compressive behavior of unconfined concrete, it is first necessary to define its peak stage properties, specifically the compressive strength (f_{c0}) and the corresponding axial strain (ε_{c0}), which serve as fundamental input parameters.

Experimental investigations (e.g., Watanabe et al. [23]) have shown that the mechanical properties of large-scale concrete columns, namely f_{c0} and ε_{c0} , are significantly lower than those obtained from standard cylindrical specimens (diameter $D = 150$ mm and height $L = 300$ mm). This discrepancy is mainly attributed to the size effect phenomenon. In the present work, the formulation proposed by Yang et al. [9] is adopted to estimate f_{c0} based on the compressive strength of standard cylinders (f'_{c0}), expressed as follows (with minor rearrangement):

$$f_{c0} = \left[0.63 + 0.9 \sqrt{\frac{\left(\frac{L}{D}\right)^{-0.6}}{1 + 0.017D}} \right] f'_{c0} \approx 1.063 \left(\frac{150}{D}\right)^{0.122} \left(\frac{D}{L}\right)^{0.088} f'_{c0} \quad (1)$$

In addition, previous experimental evidence (e.g., Dahl [28]) indicates a clear relationship between f_{c0} and the corresponding ε_{c0} with ε_{c0} generally increasing as f_{c0} increases. Beyond this dependency, Jansen Daniel and Shah Surendra [29] reported that the column aspect ratio (L/D) also plays a significant role in influencing ε_{c0} . In this study, to estimate ε_{c0} while accounting for size effects, a comprehensive database comprising 604 unconfined concrete specimens was assembled, as summarized in Table 1. Based on regression analysis of this dataset, the following empirical expression is proposed as a function of f_{c0} and the aspect ratio L/D :

$$\varepsilon_{c0} = 0.0011 \left(\frac{f_{c0} D}{L} \right)^{0.25} \quad (2)$$

The predictive capability of Eq. 2 against the collected experimental data ($\varepsilon_{c0}^{Ana}/\varepsilon_{c0}^{Exp}$) illustrated in Table 2, considering the variables f_{c0} , L/D , and D . The statistical evaluation presented in Table 2 confirms that the proposed model provides satisfactory accuracy. Moreover, its performance is superior to existing formulations proposed by Lim Jian and Ozbakkaloglu [30] and Popovics [3]. Notably, the influence of D in Eq. 2 is relatively insignificant; hence, it is neglected in the present formulation.

Table 1. Summary of the collected database.

Number of datasets		f_{c0} range (MPa)	ε_{c0} range	L/D range	D range (mm)
604	Min.	11.6	0.0012	1.0	54
	Max.	204.0	0.0051	5.5	500
	Mean	51.2	0.0025	2.2	133.4
	CoV	0.534	0.233	0.297	0.339

Table 2. Comparative performance of Eq. 2.

Model	Expression	Test Data	Assessment indicators		
			Mean	SD	MAPE
Proposed model	$\varepsilon_{c0} = 0.0011 \left(\frac{f_{c0} D}{L} \right)^{0.25}$	604	0.987	0.180	0.138
Lim Jian and Ozbakkaloglu [30]	$\varepsilon_{c0} = 0.001 f_{c0}^{0.225} \left(\frac{152}{D} \right)^{0.1} \left(\frac{2D}{L} \right)^{0.13}$	604	0.988	0.186	0.143
Popovics [3]	$\varepsilon_{c0} = 0.000937 f_{c0}^{0.25}$	604	1.009	0.192	0.147

Note: Mean = mean prediction-to-experiment ratio; SD = standard deviation of the prediction-to-experiment ratio; MAPE = mean absolute percentage error.

3. Proposed stress-strain relationship

The mechanical response of unconfined concrete under axial compression is typically characterized by a nonlinear stress-strain relationship governed by progressive microcracking, stiffness degradation, and eventual peak stress attainment. Capturing this behavior with sufficient accuracy while maintaining analytical simplicity is essential for practical constitutive modeling and subsequent structural analysis. In this study, based on observed experimental trends, a new stress-strain formulation is proposed in which the response is decomposed into two distinct stages: a nonlinear ascending branch up to the peak point and a simplified linear

descending branch thereafter.

3.1. Pre-peak stress-strain response

For the ascending branch of the stress-strain curve, a simplified polynomial function is adopted in the form:

$$\frac{f_c}{f_{c0}} = A_1 \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right)^{n_{d1}} + A_2 \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right) + A_3 \text{ for } \varepsilon_c \leq \varepsilon_{c0} \quad (3)$$

where n_{d1} defines the degree of the polynomial (non-negative integer). A_1 to A_3 are the coefficients of the polynomial. To evaluate these coefficients, the following three boundary conditions are imposed:

$$\text{I) } \frac{\varepsilon_c}{\varepsilon_{c0}} = 0 \rightarrow \frac{f_c}{f_{c0}} = 0 \quad (4)$$

$$\text{II) } \frac{\varepsilon_c}{\varepsilon_{c0}} = 1 \rightarrow \frac{f_c}{f_{c0}} = 1 \quad (5)$$

$$\text{III) } \frac{\varepsilon_c}{\varepsilon_{c0}} = 1 \rightarrow \frac{d\left(\frac{f_c}{f_{c0}}\right)}{d\left(\frac{\varepsilon_c}{\varepsilon_{c0}}\right)} = 0 \quad (6)$$

These boundary conditions are established based on the fundamental mechanical behavior of unconfined concrete under compressive loading. Eq. 6 enforces a zero-slope condition at the peak point, confirming that the derivative of the relative stress ratio with respect to the relative strain ratio equals zero at the apex. This guarantees a smooth mathematical transition from the pre-peak ascending parabola to the subsequent post-peak softening region. The coefficients of A_1 to A_3 can be, accordingly, derived as $A_1 = \frac{1}{(1-n_{d1})}$, $A_2 = -\frac{n_{d1}}{(1-n_{d1})}$ and $A_3 = 0$. Thus, Eq. 3 can be rewritten based on the boundary conditions:

$$f_c = \left[\frac{1}{1-n_{d1}} \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right)^{n_{d1}} - \frac{n_{d1}}{1-n_{d1}} \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right) \right] f_{c0} \text{ for } \varepsilon_c \leq \varepsilon_{c0} \quad (7)$$

Fig. 1a compares f_c/f_{c0} versus $\varepsilon_c/\varepsilon_{c0}$ relations reported from a set of experimental unconfined concrete specimens with a concrete strength class of 34 MPa with those obtained analytically from Eq. 7. As demonstrated in Fig. 1a, the pre-peak response of normal-strength unconfined concrete specimens, spanning concrete strength classes of 32 to 36 MPa and core dimensions (D) from 76 to 200 mm, is accurately captured by setting the polynomial degree parameter n_{d1} to 2.55. The analytical model closely matches the initial tangent stiffness and tracks the progressive, microcrack-induced nonlinearity up to the peak stress threshold (f_{c0}). This strong correlation across diverse experimental programs confirms that Eq. 7 effectively neutralizes specimen size variations when calibrated with the appropriate shape factor. Conversely, Fig. 1b evaluates the model's performance against high-strength concrete specimens ranging from 58 to 62 MPa. To accommodate the inherent mechanical differences of higher strength classes, the shape parameter n_{d1} must be increased to 4.25 based on the experimental data. From a material mechanics perspective, this elevated n_{d1} value represents a more linear stress-strain trajectory during the early-to-mid loading stages, followed by a sharper curvature change just prior to reaching peak capacity. This mathematical shift physically corresponds to the delayed microcracking and more brittle pre-peak behavior characteristic of high-strength concrete matrices, which exhibit noticeably less relative stress (f_c/f_{c0}) at given intermediate strain ratios ($\varepsilon_c/\varepsilon_{c0}$) compared to lower strength counterparts. The excellent agreement demonstrated in both figures confirms that by treating n_{d1} as a variable parameter rather than a fixed constant (such as the traditional n_{d1} adopted in classic parabolic models), the formulation preserves high predictive accuracy across distinct concrete strength spectra without losing its simplified mathematical format.

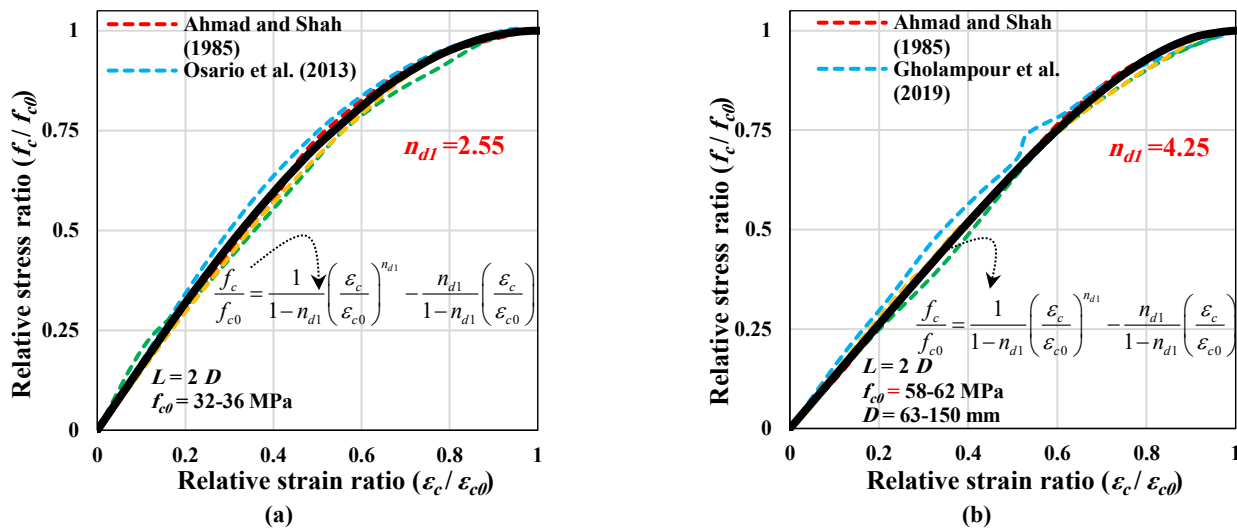


Fig. 1. Variation of f_c^{Exp}/f_{c0} with respect to $\varepsilon_c/\varepsilon_{c0}$: (a) Normal-strength concrete; and (b) High-strength concrete.

Note: Test data obtained from Ahmad and Shah [31], Dahl [28], Eid et al. [15], Osorio et al. [32], Paultre et al. [26], and Gholampour et al. [20].

In order to determine n_{d1} as an input parameter, a large test database of unconfined concrete columns was provided. For this purpose, n_{d1}^{Exp} was derived through the implementation of Eq. 7 on the experimental stress-strain relation so that Eq. 7 could closely predict the entire pre-peak stress-strain relation of counterparts. Fig. 2 presents the relations of n_{d1}^{Exp} with respect to the key variables of f_{c0} , L/D , and D . It can be seen in Fig. 2a that f_{c0} is the most significant variable in n_{d1} so that by increasing f_{c0} , n_{d1}^{Exp} increases noticeably. Fig. 2b reveals that L/D has a slight effect on n_{d1}^{Exp} . Furthermore, by increasing the normalized column dimension ($D/150$), n_{d1}^{Exp} decreases significantly (Fig. 2c).

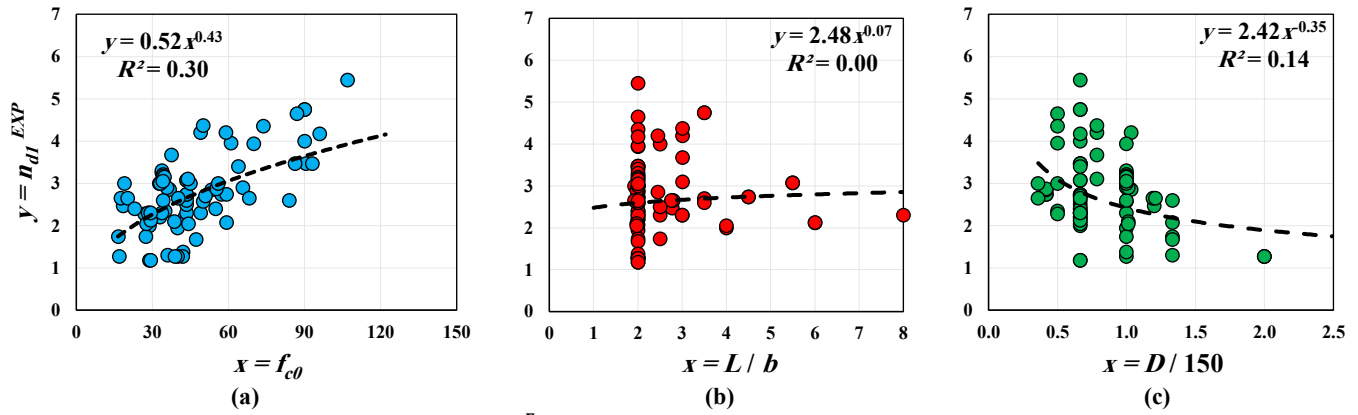


Fig. 2. Variation of n_{d1}^{Exp} with respect to: (a) f_{c0} ; (b) L/D ; and (c) $D/150$.

It is worth noting that the influence of L/D on n_{d1} is relatively weak compared with that of concrete strength. This behavior is consistent with the mechanics of the pre-peak response, during which deformation remains nearly uniform, and damage is still distributed throughout the specimen. Based on statistical analysis of the influence of key variables on n_{d1}^{Exp} , a new regression-based developed formulation is suggested as:

$$n_{d1} = \frac{0.77f_{c0}^{0.4}\left(\frac{L}{D}\right)^{0.1}}{1 + \frac{D}{300}} \quad (8)$$

Fig. 3 explicitly quantifies the predictive performance of this developed formulation by plotting the experimentally derived shape factor n_{d1}^{Exp} against the analytically calculated counterpart n_{d1}^{Ana} . The predictive reliability is statistically substantiated across 85 test data points, demonstrating a highly favorable alignment along the 1:1 ideal prediction line. The robustness of the proposed empirical equation is further confirmed by its statistical indicators, which reveal a mean ratio (Mean) of 1.047, a coefficient of variation (CoV) of 0.272, and a mean absolute percentage error (MAPE) of 0.226. The excellent agreement demonstrated in both figures confirms that by treating n_{d1} as a variable parameter determined via Eq. 8, the formulation preserves high predictive accuracy across distinct concrete strength spectra and specimen geometries without losing its simplified mathematical format.

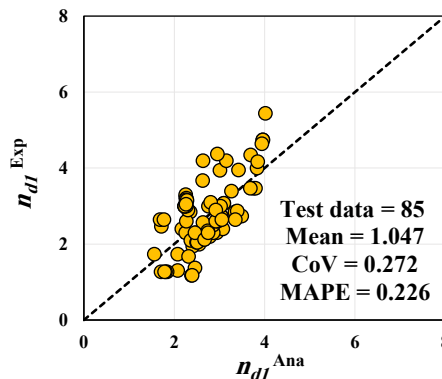


Fig. 3. Examination of n_{d1}^{Exp} versus n_{d1}^{Ana} relation.

It is noteworthy that in the case of adopting a second-degree polynomial function ($n_{d1} = 2$) to simulate the pre-peak response of unconfined concrete, by using the following three boundary conditions presented in Eqs. 4 to 6, the stress-strain relation could be determined (as proposed by Hognestad [1], Kent and Park [2], Vecchio and Collins [33], and Légeron and Paultre [6]):

$$f_c = \left[2 \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right) - \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right)^2 \right] f_{c0} \text{ for } \varepsilon_c \leq \varepsilon_{c0} \quad (9)$$

Accordingly, owing to its mathematical form, this formulation assumes a fixed normalized curvature of the ascending branch ($n_{d1} = 2$), irrespective of concrete strength or specimen geometry. Although the peak strain ε_{c0} may vary with f_{c0} , L/D , and D , modifying the peak strain only rescales the strain axis and does not alter the intrinsic evolution of the normalized stress-strain relationship. Experimental results (Figs. 2 and 3) demonstrate that these variables influence not only the peak coordinates but also the rate at which stress develops before the peak, reflecting changes in stiffness degradation associated with different material strengths and specimen geometries. Consequently, expressing their influence solely through ε_{c0} is insufficient to reproduce the experimentally observed changes in the normalized pre-peak response. Introducing n_{d1} as a variable parameter provides an

independent degree of freedom to describe these changes in the normalized constitutive response while preserving the simplicity of the polynomial formulation. Consequently, adopting a constant value of $n_{d1} = 2$ may lead to misleading predictions, whereas the proposed formulation preserves both simplicity and predictive accuracy by expressing n_{d1} as a function of the governing variables through Eq. 8.

3.2. Post-peak stress-strain response

The post-peak response of unconfined concrete under axial compression is governed by a strongly nonlinear softening behavior associated with progressive damage evolution. After reaching the peak stress, the material response is dominated by the development and coalescence of distributed microcracks into localized macro-cracks, ultimately forming a distinct fracture zone. This localization mechanism leads to a pronounced reduction in load-carrying capacity, accompanied by rapid stiffness degradation and strain concentration within the damaged region. From a continuum mechanics standpoint, deformation beyond the peak stage is no longer homogeneously distributed across the specimen. Instead, inelastic deformation becomes increasingly confined to the fracture zone, while the surrounding material primarily undergoes elastic unloading. To capture this transition in a consistent and normalized framework, a non-dimensional relative strain parameter, denoted as X_ε , is introduced. This parameter transforms the post-peak degradation process into a unified dimensionless space, facilitating generalized modeling across different concrete strengths and geometries. This ratio is strategically defined such that $X_\varepsilon = 1$ when the propagating compressive strain reaches a critical milestone of ε_{ci0} , representing the specific post-peak axial strain at which the concrete's remaining capacity drops exactly to half of its peak strength ($0.5f_{c0}$). Utilizing this boundary condition, the multi-variable relationship between the relative stress ratio (f_c/f_{c0}) and the normalized strain ratio (X_ε) is mathematically proposed via the following logistic-based type function:

$$\frac{f_c}{f_{c0}} = \frac{1}{X_\varepsilon^{n_{d2}+1}} \quad \text{for } \varepsilon_c \geq \varepsilon_{c0} \quad (10)$$

in which:

$$X_\varepsilon = \frac{\varepsilon_c - \varepsilon_{c0}}{\varepsilon_{ci0} - \varepsilon_{c0}} \quad (11)$$

From a material mechanics and continuum perspective, the milestone $X_\varepsilon = 1$ (where $\varepsilon_c = \varepsilon_{c0}$) physically isolates the mathematical inflection point of this logistic sigmoidal function. At this stage, the response reaches $0.5f_{c0}$, which physically represents the point of maximum degradation rate, where the slope $df_c/d\varepsilon$ attains its most negative value. This inflection behavior captures the transition from distributed damage to highly localized fracture-controlled softening. To accommodate distinct material characteristics, the exponent n_{d2} is introduced as an empirical shape-adjustment parameter. This parameter governs the curvature and steepness of the post-peak branch, enabling the model to adapt to different concrete strengths and specimen aspect ratios. In this way, n_{d2} controls the rate of strain softening and indirectly reflects changes in fracture energy dissipation related to varying material configurations.

Fig. 4a presents the variation of the normalized stress ratio f_{c0} with respect to X_ε , obtained from unconfined concrete specimens with an average concrete strength of 62 MPa and a constant slenderness ratio of $L/D = 2$, but with different cross-sectional dimensions D . As shown, the proposed model in Eq. 10 reproduces the experimental softening response with good agreement when the shape parameter is calibrated as $n_{d2} = 1.88$. In this case, the model captures both the initial steep post-peak drop and the subsequent gradual decay in residual capacity, indicating that the logistic formulation is capable of representing the transition from distributed damage to localized fracture. Fig. 4b shows the influence of concrete strength on the post-peak curvature. For the lower strength class of 34 MPa, the experimental response exhibits a less pronounced curvature and a more gradual reduction in stress. This reflects a more distributed cracking process and higher strain capacity prior to complete localization. Accordingly, a lower value of $n_{d2} = 1.62$ is required to reproduce this smoother softening response. The reduction in n_{d2} with decreasing strength is consistent with the increased ductility and delayed localization typically observed in lower-strength concretes. The effect of geometry is examined in Fig. 4c in terms of the column aspect ratio L/D . Increasing L/D leads to a more pronounced post-peak softening response, associated with a stronger tendency toward strain localization over a shorter effective crushing zone. This behavior is reflected in a systematic increase of the calibrated parameter n_{d2} , from 1.47 for $L/D = 2$ to 2.91 for $L/D = 4$. The increase in n_{d2} indicates a steeper decay in stress after peak, consistent with reduced confinement stability and more abrupt fracture development in slender specimens. Overall, Eq. 10 provides a consistent representation of the post-peak behavior of unconfined concrete across different strength levels and geometries. The model effectively links the curvature of the softening branch to both material properties and structural slenderness through the parameter n_{d2} . In this study, both ε_{ci0} and n_{d2} were identified through back-analysis by fitting Eq. 10 to the experimental stress-strain curves to obtain parameter sets that best reproduce the observed post-peak response.

To fully define the proposed logistic-based softening formulation, the two governing parameters controlling the location and shape of the post-peak branch, namely the normalized half-capacity strain ratio ($\varepsilon_{ci0}/\varepsilon_{c0}$) and the curvature parameter (n_{d2}), must be expressed as functions of measurable material and geometric properties. Accordingly, the experimentally calibrated values of ε_{ci0} and n_{d2} , obtained through the back-analysis procedure described previously, were systematically examined using the complete database of unconfined concrete specimens. Particular attention was given to the influence of f_{c0} , L/D , and D , which are known to influence fracture localization, size effect, and post-peak deformation capacity.

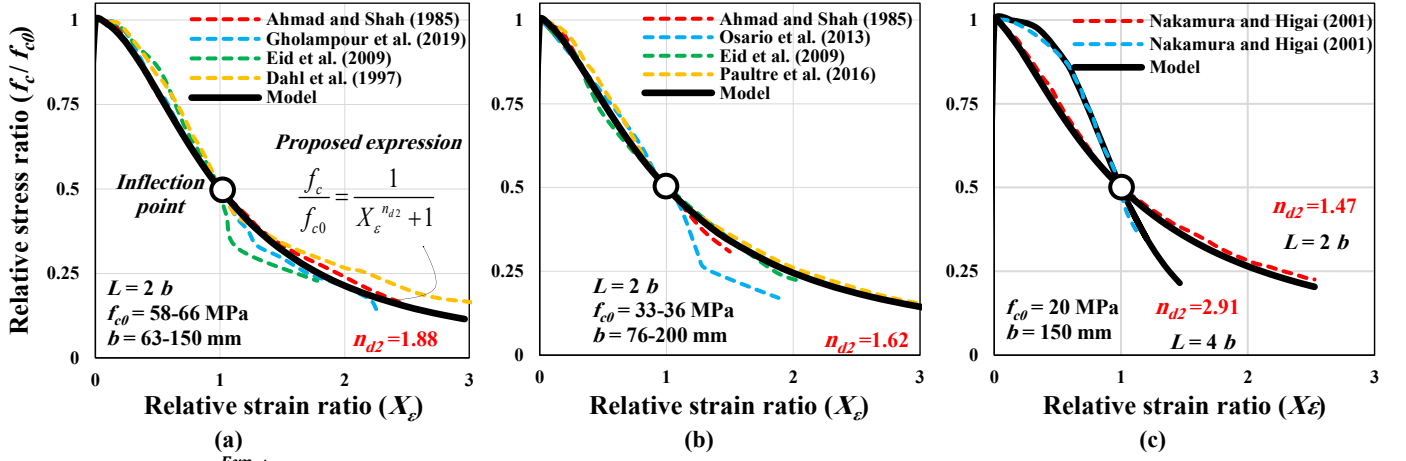


Fig. 4. Variation of f_c^{Exp}/f_{c0} with respect to X_ϵ : (a) effect of specimen size; (b) effect of concrete compressive strength; and (c) effect of specimen aspect ratio.

Note: Test data obtained from Ahmad and Shah [31], Dahl [28], Nakamura and Higai [34], Eid et al. [15], Osorio et al. [32], Paultre et al. [26], and Gholampour et al. [20].

Fig. 5 presents the variation of the experimentally extracted strain ratio ($\epsilon_{ci0}/\epsilon_{c0}$) with respect to the considered variables. As shown in Fig. 5a, a clear decreasing trend is observed with increasing concrete strength. Although the database exhibits the typical scatter associated with post-peak measurements, the overall power-law trend remains evident. This behavior is consistent with the fracture characteristics of higher-strength concrete, which generally possesses a denser microstructure and reduced crack tortuosity. Consequently, once peak stress is attained, crack propagation becomes more localized and unstable, causing the stress to decline to 50% of its peak value within a relatively smaller strain interval. As a result, the $\epsilon_{ci0}/\epsilon_{c0}$ ratio decreases with increasing strength. A similar trend can be observed in Fig. 5b with respect to the aspect ratio (L/D), where the relation indicates a pronounced reduction in $\epsilon_{ci0}/\epsilon_{c0}$ as the specimen becomes more slender. From a mechanics standpoint, post-peak deformation in unconfined concrete is governed primarily by strain localization within a relatively short fracture process zone. While this localized region undergoes significant inelastic deformation, the remaining portion of the specimen experiences progressive elastic unloading. Therefore, as L/D increases, the localized deformation becomes distributed over a larger gauge length, reducing the apparent average ductility measured at the specimen level and consequently lowering the normalized half-capacity strain ratio. The influence of specimen size is illustrated in Fig. 5c. When considered independently, the normalized dimension ($D/150$) exhibits a relatively weak positive correlation. However, the low coefficient of determination indicates that the apparent trend is strongly influenced by interactions with the other governing variables. In fact, when the complete database is analyzed simultaneously through multi-variable regression, the size effect contribution becomes negative. This outcome is physically reasonable, as larger specimens contain a greater volume of material defects and microstructural heterogeneities, promoting earlier crack coalescence and more rapid localization after peak stress. Consequently, the strain required to reach 50% of the peak stress tends to decrease with increasing specimen size.

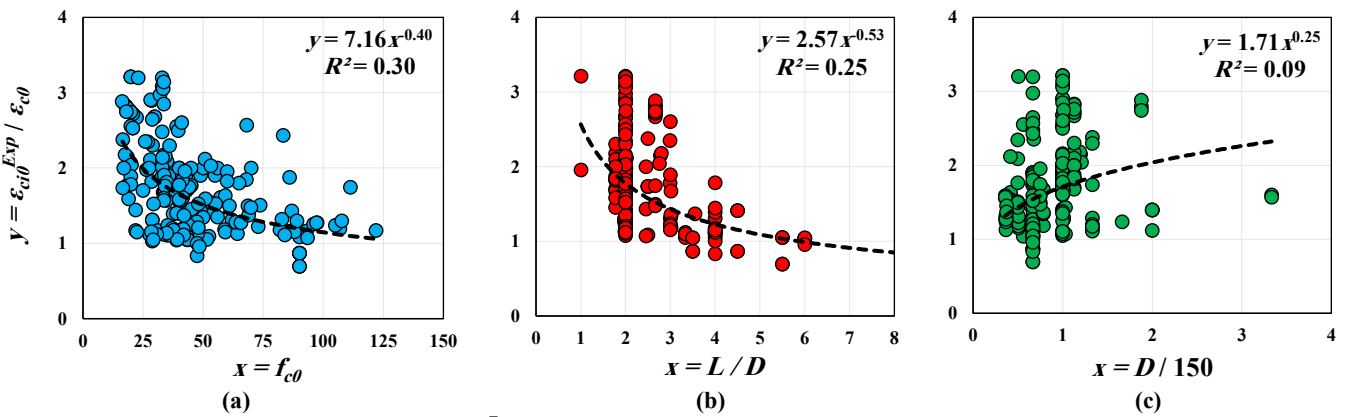


Fig. 5. Variation of $\epsilon_{ci0}^{Exp}/\epsilon_{c0}$ with respect to: (a) f_{c0} ; (b) L/D ; and (c) $D/150$.

multi-variable regression analysis was conducted by coupling the cumulative dependencies of all three variables simultaneously. By prioritizing the coupled power-law interactions, based on regression analysis, the predictive formulation for the $\epsilon_{ci0}^{Exp}/\epsilon_{c0}$ ratio can be obtained as:

$$\frac{\epsilon_{ci0}}{\epsilon_{c0}} = 23.4 f_{c0}^{-0.59} \left(\frac{L}{D}\right)^{-0.56} \left(\frac{D}{150}\right)^{-0.1} \quad (12)$$

To rigorously confirm the predictive accuracy of the proposed multi-variable empirical formulation for $\epsilon_{ci0}/\epsilon_{c0}$ ratio, the analytical results calculated via Eq. 12 are directly verified against 217 distinct experimental test data points in Fig. 6. The data points cluster tightly along the 1:1 ideal equality line, yielding an exceptional Mean value of 0.987, which demonstrates that the formulation converges to the mechanical center of the experimental cloud with negligible systematic bias. Furthermore, despite the

high inherent scatter and chaotic macro-cracking kinetics characteristic of unconfined concrete strain-softening, the model maintains a robust CoV of 0.210 and a MAPE of 0.160. This high degree of convergence scientifically validates the simultaneous power-law coupling of f_{c0} , L/D and $D/150$, proving the suitable performance of Eq. 12.

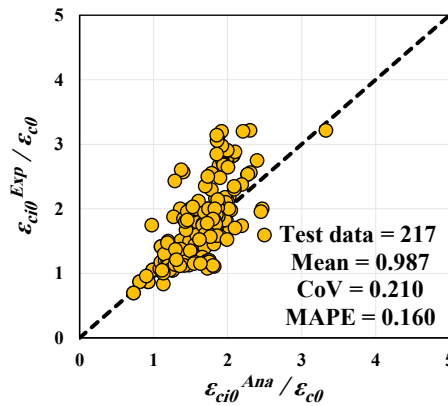


Fig. 6. Predictive performance of Eq. 12.

The experimentally calibrated values of the post-peak curvature parameter (n_{d2}^{Exp}) are examined in Fig. 7. Unlike $\varepsilon_{ci0}/\varepsilon_{c0}$, which primarily controls the position of the softening branch, n_{d2} governs its curvature and therefore directly influences the rate of stress degradation following peak loading. As shown in Fig. 7a, the influence of concrete strength is relatively modest. Nevertheless, the observed trend indicates that increasing strength modifies the curvature of the softening branch. Nevertheless, the observed trend suggests that increasing concrete strength systematically modifies the shape of the post-peak response. Since higher-strength concretes typically experience more rapid crack localization and reduced post-peak deformation capacity, a substantial portion of the strength loss occurs within a relatively narrow strain interval immediately after peak stress.

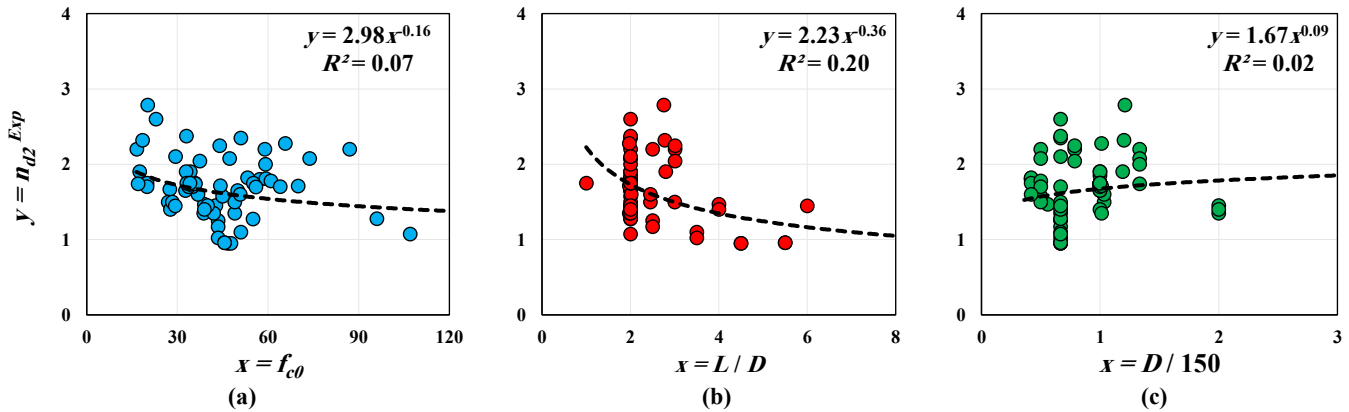


Fig. 7. Variation of n_{d2}^{Exp} with respect to: (a) f_{c0} ; (b) L/D ; and (c) $D/150$.

Consequently, the softening branch reaches its inflection region earlier and exhibits a less extended transition zone. Within the proposed logistic formulation, this behavior is captured through a reduction in n_{d2} , indicating that the curvature of the normalized softening response is sensitive to the degree of brittleness associated with increasing concrete strength. Fig. 7b demonstrates that the aspect ratio (L/D) is the most influential variable governing n_{d2} . From a mechanical standpoint, post-peak deformation in unconfined concrete is concentrated within a localized fracture zone whose characteristic length is substantially smaller than the overall specimen height. As the aspect ratio increases, the ratio between the fracture-zone length and the specimen length decreases, causing the inelastic deformation to become increasingly localized relative to the total specimen dimension. Consequently, a larger proportion of the specimen undergoes elastic unloading while only a limited region continues to accumulate damage and inelastic strains. This mechanism produces a more abrupt transition from distributed damage to localized crushing, resulting in a sharper change in the slope of the stress-strain curve immediately after the peak point. In the normalized space adopted by the proposed model, this behavior manifests as an increase in the curvature of the softening branch around the inflection region. Therefore, larger aspect ratios require systematic adjustment of n_{d2} to reproduce the experimentally observed localization-induced stress degradation. The strong statistical correlation observed in Fig. 7b confirms that the evolution of post-peak curvature is governed primarily by geometric effects associated with strain localization rather than by material strength alone. The influence of normalized specimen size is presented in Fig. 7c. Although a slight positive trend is observed, it indicates a negligible contribution. Therefore, inclusion of this parameter in the final model provides little improvement in predictive accuracy while increasing model complexity. Consequently, the size term was excluded from the final regression formulation.

Based on the preceding observations, a simplified multi-variable regression analysis was conducted using the two dominant variables, namely concrete compressive strength and specimen aspect ratio. The resulting expression is given by:

$$n_{d2} = 4.2f_{c0}^{-0.17} \left(\frac{L}{D}\right)^{-0.36} \quad (13)$$

To assess the predictive capability of the proposed expression for the post-peak curvature parameter n_{d2} , the analytical values calculated using Eq. 13 are compared against 71 experimentally calibrated values obtained through back-analysis, as shown in Fig. 8. Overall, the predictions exhibit a close agreement with the experimental counterparts, with the data points distributed reasonably around the 1:1 line and without any noticeable systematic bias over the considered range of parameters. The statistical indicators further confirm the reliability of the proposed formulation. A mean value of 1.030 indicates that Eq. 13 provides nearly unbiased predictions, with only a slight tendency toward conservative estimation. In addition, the relatively low CoV = 0.195 and MAPE = 0.165 demonstrate that the scatter between predicted and experimental values remains limited despite the inherent variability associated with post-peak concrete behavior. Considering that n_{d2} governs the curvature of the softening branch and is therefore influenced by complex interactions between material properties, strain localization, and fracture development, the obtained level of accuracy is considered satisfactory. The results suggest that the combined influence of f_{c0} and L/D , as incorporated in Eq. 13, captures the primary mechanisms controlling the evolution of post-peak curvature. Moreover, the exclusion of D avoids unnecessary model complexity while maintaining a high level of predictive performance, thereby resulting in a statistically robust and practically applicable formulation.

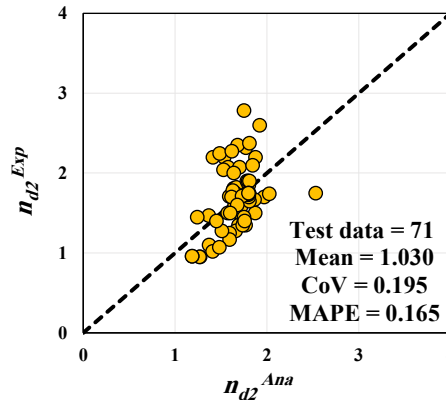


Fig. 8. Predictive performance of Eq. 13.

4. Validation of the proposed stress-strain relationship

To assess the predictive capability of the proposed stress-strain model, independent experimental results reported in the literature were selected for validation. The comparison focuses on the model's ability to reproduce the influence of the governing parameters identified during the calibration process, namely concrete compressive strength and specimen aspect ratio, on both the pre-peak and post-peak responses of unconfined concrete. Before presenting the validation results, the complete formulation of the proposed model is summarized in Table 3. It should be noted that each mathematical form was selected according to the characteristics of the response being represented. Power-law formulations were adopted for the empirical parameter relationships because they provide a compact and robust description of the nonlinear dependence of the model parameters on concrete strength and specimen geometry using only a few coefficients. The polynomial function was adopted for the ascending branch because it satisfies the required constitutive boundary conditions while allowing the curve shape to be governed by a single parameter. The logistic function was selected for the descending branch because it naturally represents the progressive strain-softening process through a smooth transition and a physically meaningful inflection point associated with the onset of damage localization.

Table 3. Summary of the proposed model.

Stress-strain relationship:

$$f_c = \left[\frac{1}{1 - n_{d1}} \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right)^{n_{d1}} - \frac{n_{d1}}{1 - n_{d1}} \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right) \right] f_{c0} \quad \text{for } \varepsilon_c \leq \varepsilon_{c0}$$

$$\frac{f_c}{f_{c0}} = \frac{1}{\left(\frac{\varepsilon_c - \varepsilon_{c0}}{\varepsilon_{ci0} - \varepsilon_{c0}} \right)^{n_{d2}} + 1} \quad \text{for } \varepsilon_c \geq \varepsilon_{c0}$$

in which:

$$\varepsilon_{c0} = 0.0011 \left(\frac{f_{c0} D}{L} \right)^{0.25}; \quad \frac{\varepsilon_{ci0}}{\varepsilon_{c0}} = 23.4 f_{c0}^{-0.59} \left(\frac{L}{D} \right)^{-0.56} \left(\frac{D}{150} \right)^{-0.1}; \quad n_{d1} = \frac{0.77 f_{c0}^{0.4} \left(\frac{L}{D} \right)^{0.1}}{1 + \frac{D}{300}}; \quad n_{d2} = 4.2 f_{c0}^{-0.17} \left(\frac{L}{D} \right)^{-0.36}$$

Fig. 9a compares the analytical predictions with the experimental stress-strain curves reported by Wischers [35] for unconfined concrete specimens with different concrete strength levels. As shown, the proposed model accurately captures the progressive change in stress-strain behavior associated with increasing compressive strength. In particular, the model successfully reproduces the increase in peak stress, the corresponding variation in peak strain, and the changes in the post-peak softening response. The agreement between analytical and experimental results indicates that the proposed formulations are capable of appropriately representing the influence of material strength on both the ascending and descending branches of the stress-strain curve. Moreover, the model correctly predicts the tendency of higher-strength concrete to exhibit a more localized and less ductile post-peak response compared with lower-strength concrete. The influence of specimen geometry is examined in Fig. 9b using the experimental results reported by Jansen Daniel and Shah Surendra [29]. The considered specimens possess similar material properties but different aspect ratios (L/D). As demonstrated, the proposed model adequately reproduces the observed effect of specimen slenderness on the stress-

strain response, particularly in the post-peak region where geometric effects become increasingly significant. The analytical curves correctly capture the reduction in deformation capacity and the modification of the softening branch associated with increasing aspect ratio. This behavior is consistent with the underlying assumption that post-peak deformation is governed by strain localization within a fracture zone whose relative contribution decreases as the specimen length increases. Overall, the comparisons presented in Fig. 9 demonstrate that the proposed model provides a realistic representation of the complete stress-strain response of unconfined concrete over a wide range of material strengths and geometric configurations. The ability of the model to accurately reproduce both the peak-stage characteristics and the post-peak softening behavior confirms the robustness of the proposed formulations and supports their applicability for analytical and numerical modeling of concrete columns.

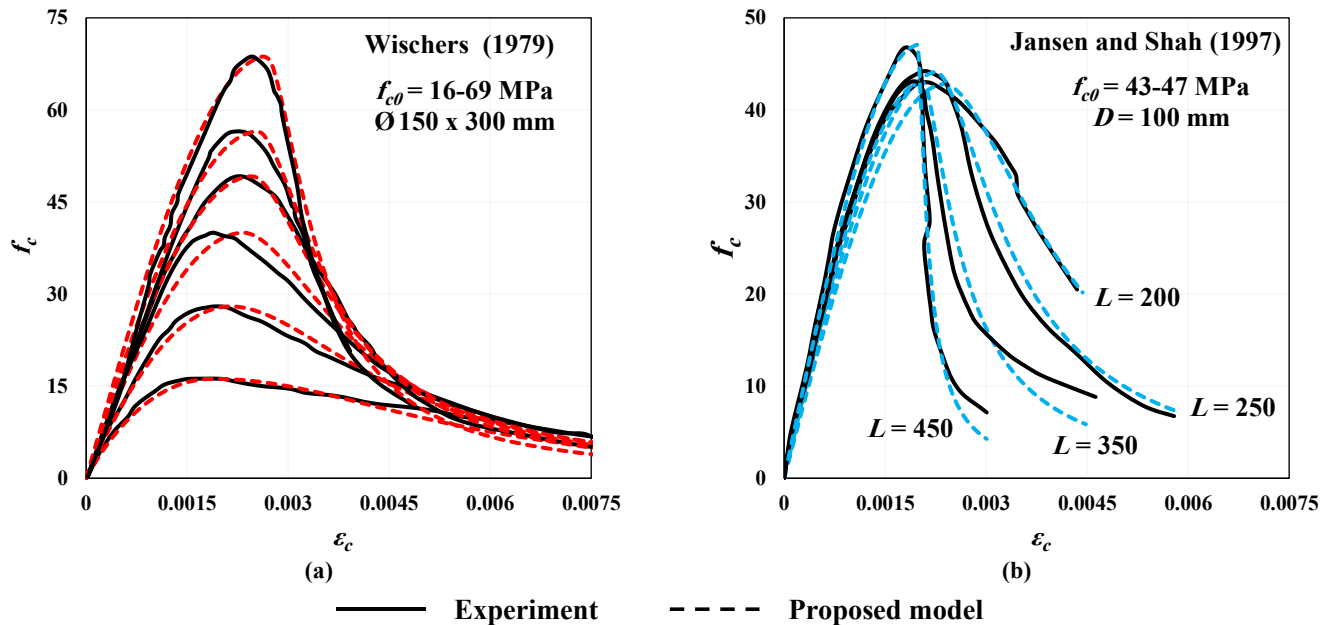


Fig. 9. Comparison of the results obtained from the experiments and the proposed model: (a) effect of concrete compressive strength; and (b) effect of specimen aspect ratio.

To rigorously demonstrate the global reliability, versatility, and predictive stability of the proposed stress-strain model, the analytical response curves are validated against a comprehensive set of independent experimental results, as compiled in Fig. 10. The experimental stress-strain data used for this validation were collected from thirteen independent historical research programs, namely those conducted by Ahmad and Shah [31], Dahl [28], Taerwe [36], Jansen Daniel and Shah Surendra [29], Nakamura and Higai [34], Watanabe et al. [23], Eid et al. [15], Desnerck et al. [37], Osorio et al. [32], Lim and Ozbakkaloglu [8], Hany Najwa et al. [38], Paultre et al. [26], and Gholampour et al. [20]. This extensive database ensures that the validation covers a wide spectrum of testing conditions and material characteristics.

As shown across all subfigures in Fig. 10, the proposed analytical model exhibits strong predictive capability by simultaneously capturing both the nonlinear ascending pre-peak response and the highly nonlinear post-peak softening behavior across all considered experimental datasets. The validation matrix spans a broad range of unconfined concrete column configurations, including varying geometric aspect ratios (L/D), cross-sectional dimensions (D), and concrete compressive strengths ranging from normal-strength concrete (17 MPa) to high-strength concrete (107 MPa). For high-strength specimens (e.g., $f_{c0} = 90, 94$, and 107 MPa), the model accurately reproduces the characteristic brittle response, including a nearly linear pre-peak branch followed by a rapid and pronounced post-peak stress degradation. This behavior is consistent with the reduced deformation capacity and enhanced strain localization typically observed in high-strength cementitious materials, which is effectively reflected by the proposed formulation. In contrast, for normal- and low-strength concrete specimens (e.g., $f_{c0} = 17$ and 20 MPa), the analytical model accurately captures the more gradual transition from peak stress to residual capacity, characterized by a smoother softening response and extended post-peak deformation range. This reflects the more distributed cracking process and higher energy dissipation capacity associated with lower-strength concrete matrices. Importantly, the model reproduces these contrasting behavioral regimes in a continuous and unified manner, without requiring case-specific recalibration of parameters. Furthermore, the validation confirms that the model remains robust across a wide range of specimen geometries, including cylinder diameters from $\varnothing 63$ mm to $\varnothing 200$ mm and specimen lengths up to 600 mm. The consistent agreement between analytical predictions and experimental observations across these diverse configurations demonstrates that the embedded size-dependent peak formulations, together with the calibrated shape parameters ($\epsilon_{c0}, \epsilon_{ci0}, n_{d1}$ and n_{d2}), effectively capture both global scaling effects and localized macro-cracking mechanisms governing post-peak behavior. Overall, the close correspondence observed throughout Fig. 10 confirms that the proposed model provides a unified and physically consistent representation of the full compressive response of unconfined concrete. The demonstrated accuracy across a broad range of material strengths, geometric properties, and experimental programs further establishes its reliability as a general-purpose constitutive tool, applicable without the need for case-specific recalibration.

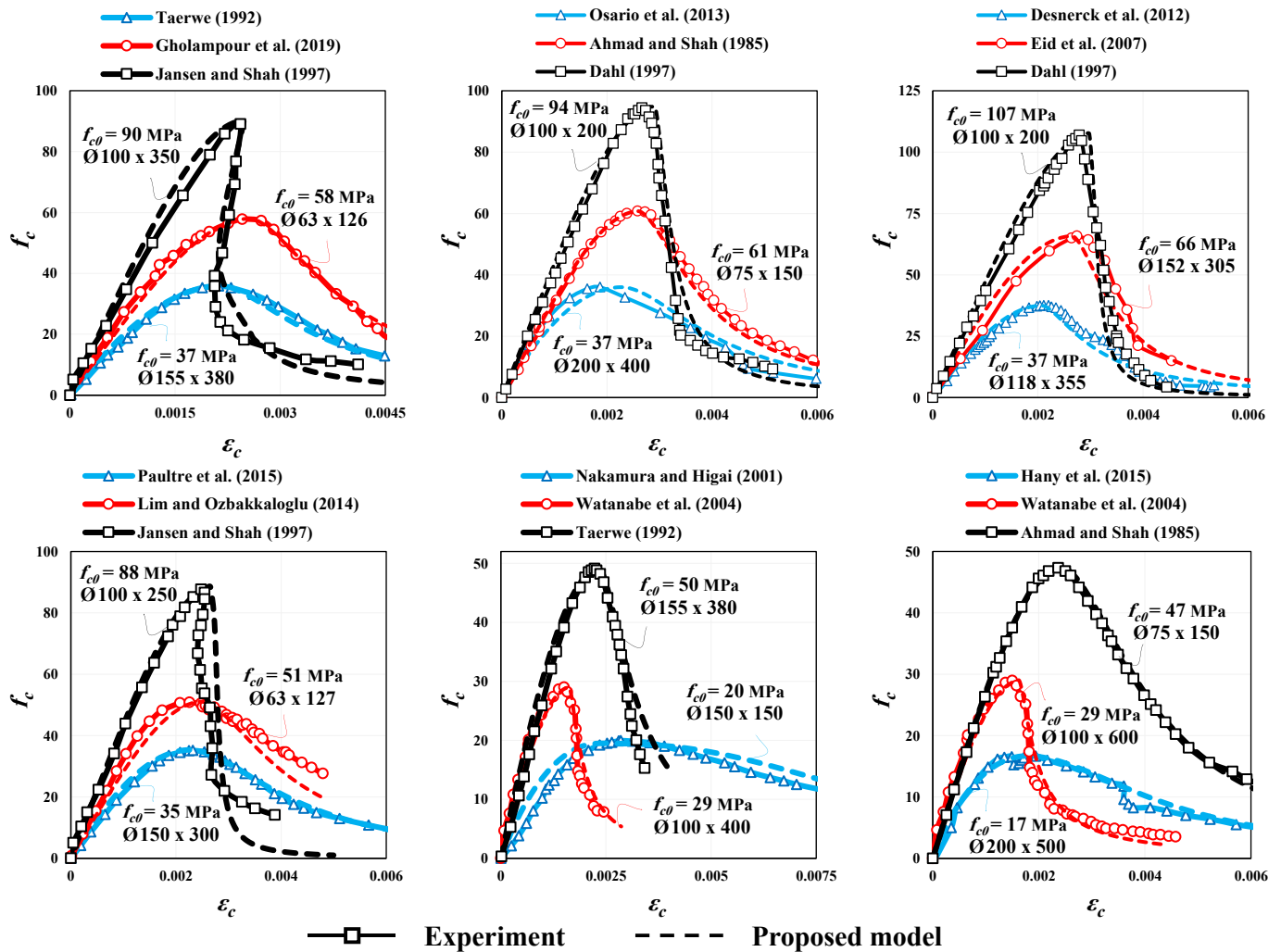


Fig. 10. Comparison between results obtained from the experiments and the proposed model.

5. Conclusions

This study proposed a unified size-dependent stress-strain model for unconfined concrete under monotonic axial compression. The formulation was established through regression analysis of a comprehensive experimental database and validated against independent experimental stress-strain curves covering a broad range of concrete strengths and specimen geometries. By integrating simplified empirical relationships with physically based constitutive functions, the proposed model provides a practical representation of both the ascending and descending branches of the compressive response. Based on the results obtained, the main findings of this study are summarized as follows:

- A unified constitutive formulation was developed that explicitly incorporates the effects of concrete strength, specimen size, and aspect ratio, enabling consistent prediction of the compressive response of both normal- and high-strength unconfined concrete.
- New regression-based expressions were established for the peak strain and the governing pre-peak and post-peak parameters, providing a simple yet robust representation of their dependence on material strength and specimen geometry.
- Introducing a variable pre-peak shape parameter allows the model to capture experimentally observed changes in the normalized ascending response that cannot be represented by conventional formulations employing a fixed second-degree parabola.
- The proposed two-branch formulation accurately reproduces both the ascending and descending branches of the compressive stress-strain response while maintaining mathematical simplicity and computational efficiency.
- Validation against independent experimental stress-strain curves demonstrated that the proposed model reliably predicts the complete compressive response over a broad range of concrete strengths and specimen geometries.

Overall, the results demonstrate that the proposed stress-strain model provides a physically consistent and computationally efficient representation of the complete compressive response of unconfined concrete. By explicitly incorporating size effects and fracture localization into a compact analytical framework, the model offers a practical constitutive relationship for nonlinear structural analysis and performance-based assessment of reinforced concrete structures (e.g., Shayanfar and Akbarzadeh Bengar [39-41], Shayanfar et al. [10, 42-44], and Dashtaki et al. [45]). In particular, the proposed formulation can be readily implemented in nonlinear finite element models and other computational analysis frameworks to improve the simulation of concrete members

with different strength classes and geometric configurations (e.g., Nematzadeh et al. [46], Shayanfar et al. [47], Tariq et al. [48], and Khant et al. [49]), thereby supporting more reliable structural assessment and design.

It should be noted that the proposed formulation is applicable to the monotonic compressive response of unconfined concrete. Extension to cyclic loading is beyond the scope of the present study and would require coupling the proposed monotonic envelope with appropriate unloading-reloading rules and a damage evolution law capable of representing stiffness degradation, strength deterioration, and irreversible strain accumulation under repeated loading. This represents a promising direction for future research.

Statements & declarations

Author contributions

Javad Shayanfar: Conceptualization, Methodology, Data curation, Software, Validation, Writing - Original Draft.

Joaquim A. O. Barros: Conceptualization, Methodology, Project administration, Resources, Writing - Review & Editing.

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Data availability

All data are available in the manuscript.

Declarations

The authors declare no conflict of interest.

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Comparative Blast Performance of Arched and Three-Layer Space-Frame Shields for Offshore Platforms

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ABSTRACT

Accidental gas explosions on fixed offshore jacket platforms can impose short-duration, high-intensity loads on safety-critical topside and primary structural components. This study evaluates two lightweight blast-shield concepts intended to reduce structural demand: a convex arched double-layer space frame that promotes shock-wave deflection and a flat three-layer space frame that increases out-of-plane stiffness and member stability. Three shield configurations were generated using formex-based topology processing and analyzed in Abaqus/Explicit under a standardized 2000 kg TNT-equivalent airblast at an 8.0 m stand-off distance scaled distance $Z = 0.63 \text{ m/kg}^{1/3}$. Relative to the flat double-layer baseline, the arched shield reduced internal strain energy by approximately 25% (from 7215 to 5400 kJ), peak transmitted base shear by approximately 23% (from 1100 to 850 kN), and maximum central displacement by approximately 30% (from 2.33 to 1.62 m). The three-layer configuration provided the greatest deformation control, reducing the maximum central displacement by approximately 51% to 1.15 m, while its additional structural depth improved member-stability potential. The three-layer configuration limited the maximum central displacement to 1.15 m compared with 2.33 m for the baseline and reduced the characteristic unbraced length of the internal diagonal members, thereby improving their member-stability potential. The results indicate that curvature is effective when weight and force transmission govern design, whereas the three-layer topology is preferable when displacement and local stability are controlling criteria. These findings are specific to the modeled geometry, material assumptions, and blast scenario and should be validated experimentally before design application. These results demonstrate a criterion-dependent trade-off: the arched configuration is advantageous when reduced structural demand and support-force transmission are prioritized, whereas the three-layer configuration is preferable when deformation control and member stability govern. The findings are scenario-specific and should be interpreted within the limitations of the adopted CONWEP-based numerical framework.

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1. Introduction

Fixed offshore jacket platforms support hydrocarbon extraction, processing, and temporary storage in congested environments where structural, process, and safety systems are closely integrated. In addition to wave, wind, current, and seismic actions, accidental gas explosions and subsequent fires can impose severe transient demands on topside modules and primary load-bearing members. The rapid pressure rise associated with an explosion may cause local plate deformation, connection failure, member

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buckling, and, in extreme cases, progressive collapse [1-6].

Conventional protection systems include corrugated steel blast walls, profiled plates, and heavily stiffened panels. These systems can protect living quarters and critical process modules, but large-scale application increases topside mass and may transfer substantial reaction forces to the supporting jacket. Flat panels are also vulnerable to out-of-plane deformation, local stiffener buckling, and tearing at welded boundaries under high impulsive pressure [7-20].

Lightweight sandwich concepts with honeycomb, auxetic, foam, or composite cores have been proposed to improve specific energy absorption. Their performance can be attractive for localized panels, although manufacturing complexity, connection design, inspection, and the need for secondary support frames may limit their application over the full height of an offshore platform [21-30].

A spatial truss offers an alternative load path. Double-layer space frames provide three-dimensional redundancy and distribute localized pressure reactions to major platform joints. When combined with a ductile low-yield-point (LYP) steel cladding, the outer plate can dissipate energy through membrane yielding while the space frame spreads the residual load. Formex algebra also permits systematic generation and pruning of spatial grids, allowing weight and stiffness to be adjusted without manually rebuilding each topology [31].

Nevertheless, a flat shield remains exposed to near-normal reflection over its central region, where the reflected pressure is greatest. Curved protective surfaces may reduce the effective normal blast demand by increasing local incidence obliquity away from the apex, providing a potential geometric mechanism for reducing the load transmitted into the supporting structure. Curvature therefore has the potential to reduce the energy entering the structure rather than relying exclusively on plastic absorption. Related offshore analyses have largely focused on flat blast walls, corrugations, sandwich panels, and global platform response [32-47].

1.1. Historical context and design requirements

Major offshore accidents have demonstrated that inadequate separation and protection between process areas, escape routes, and accommodation modules can amplify the consequences of fire and explosion. Modern accidental-limit-state design therefore requires the load path, ductility, connection capacity, and post-event stability of safety-critical structures to be assessed explicitly. General blast-resistant design guidance and the Iranian code for space structures provide the broader design context adopted in this study [31, 42, 48].

Recent studies have further advanced the numerical and design assessment of blast-resistant systems. Kim et al. [30] investigated the mechanical response of hexagonal honeycomb sandwich blast walls for offshore applications and developed a numerical-based design approach, while Kim and Sohn [49] examined the deflection response of corrugated blast walls over a broad range of geometric and loading conditions. More recently, Li et al. [50] investigated the nonlinear dynamic response and damage of an offshore platform subjected to flammable-gas explosions, including the effects of explosion intensity and source location, whereas Lee and Seo examined the dynamic response of corrugated blast walls under hydrogen blast loading. These recent developments highlight the continuing importance of structural configuration, blast scenario, and nonlinear numerical modeling in the design and assessment of offshore protective systems [30, 49-51].

1.2. Dynamic response to impulsive loading

Explosion loading is characterized by a rapid rise and short positive phase. Structural response is therefore controlled by inertia, stress-wave propagation, strain-rate sensitivity, and the ratio of load duration to the relevant natural periods. Static-equivalent analysis may not capture higher-mode excitation or transient force reversals; an explicit dynamic solution is consequently appropriate for the present problem.

1.3. Low-yield-point steel cladding

LYP steel was selected for the sacrificial cladding because its relatively low yield threshold and high ductility promote early membrane yielding and distributed plastic deformation. Accordingly, the 3.5 mm reference thickness was selected to maintain this sacrificial response while limiting additional mass and stiffness.

1.4. Research gap and objectives

Overall, previous offshore blast-protection studies have predominantly focused on conventional stiffened or corrugated blast walls, sandwich and composite panels, blast-wall arrangement, and the global response of offshore structures [7, 8, 11, 21, 32, 39]. In contrast, comparatively limited attention has been given to the direct comparison of alternative space-frame shield configurations under identical loading and support conditions. The present study addresses this gap by systematically comparing flat double-layer, arched double-layer, and three-layer space-frame shields in terms of blast demand, plastic response, deformation, member stability, energy response, and transmitted support reactions.

Previous offshore blast-protection studies have predominantly concentrated on conventional flat blast walls, corrugated panels, sandwich or cellular systems, and the global response of offshore structures. Although these approaches have established important principles for blast resistance, comparatively limited attention has been devoted to large space-frame shields in which geometric curvature or an additional structural layer is used to modify the load path and transient response. In particular, direct comparisons

of flat double-layer, curved double-layer, and three-layer space-frame shields under identical blast, material, and support conditions remain limited. Moreover, the trade-off between shield-level deformation control and the reactions transmitted to the supporting offshore structure has received relatively little attention.

Accordingly, the present study provides a comparative numerical assessment of three predefined configurations: (I) a flat double-layer baseline, (II) a convex arched double-layer shield, and (III) a flat three-layer shield. Their performance is compared in terms of incidence-angle-dependent blast demand, equivalent plastic strain, internal strain energy, central displacement, member-stability response, and transmitted base shear under a common severe airblast condition. The objective is therefore to identify the configuration-dependent trade-offs relevant to offshore blast-shield selection rather than to establish a universal optimum topology.

The novelty of the present work therefore lies not in proposing a new optimization algorithm or analytical formulation, but in providing a controlled comparative assessment of three distinct space-frame blast-shield configurations under identical loading, material, and support conditions. By simultaneously examining blast demand, PEEQ, displacement, internal strain energy, member-stability response, and transmitted base shear, the study identifies the different performance trade-offs associated with geometric curvature and additional structural depth and provides a more direct basis for configuration selection in offshore blast protection (Fig. 1).

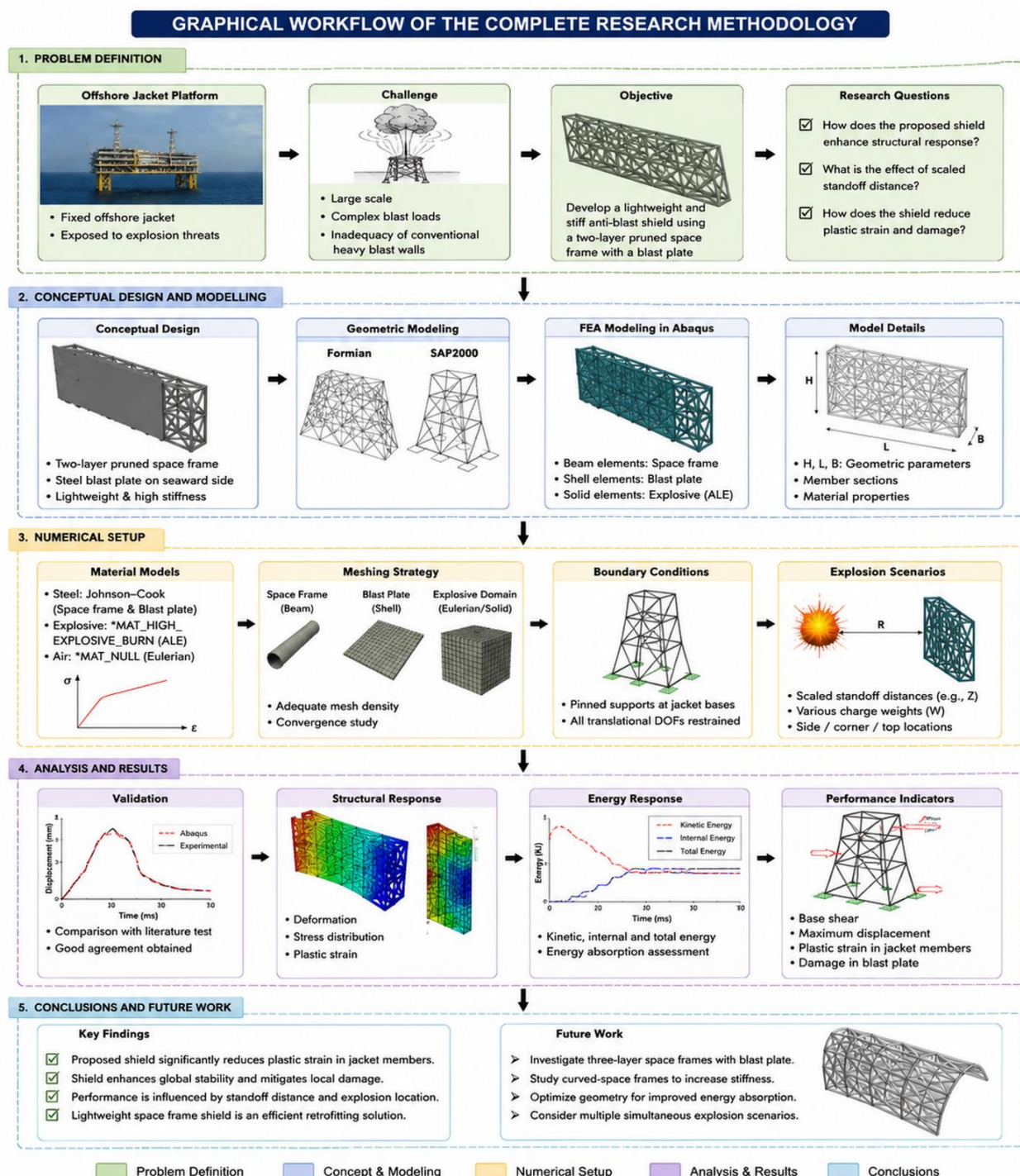


Fig. 1. Graphical workflow of the research.

2. Blast-load representation

2.1. Blast-load representation

The accidental gas explosion was represented by an equivalent TNT airblast so that all three structural configurations could be compared under an identical pressure field. The free-field positive phase was described using the modified Friedlander waveform (Eq. 1).

$$P_s(t) = P_{so}[1 - (t - t_a)/t_d] \exp[-b(t - t_a)/t_d] \quad (1)$$

where $P_s(t)$ is the incident overpressure, P_{so} is the peak incident overpressure, t_a is the arrival time, t_d is the positive-phase duration, and b is a waveform-decay coefficient.

For a surface normal to the incident shock and air with a specific-heat ratio of approximately 1.4, the normal reflected overpressure may be expressed as Eq. 2.

$$P_r = 2P_{so}(7P_0 + 4P_{so})/(7P_0 + P_{so}) \quad (2)$$

where P_r is the reflected overpressure and P_0 is the ambient pressure. The strong-shock limit of Eq. 2 approaches eight times the incident overpressure. On the arched surface, the angle between the incident wave direction and the local outward normal varies spatially; the CONWEP implementation evaluates the pressure history at each loaded surface location using its distance and incidence angle [42, 48]. Although CONWEP provides a computationally efficient representation of incident and reflected blast loading, it does not explicitly resolve the surrounding air domain or the evolving blast-wave field. Consequently, detailed diffraction, multiple reflections between structural components, shielding and confinement effects, and two-way fluid-structure interaction are not directly captured. These simplifications may affect the absolute pressure and impulse distributions and therefore the predicted deformation, plastic response, and support reactions. Their influence may be particularly relevant for the arched configuration because its predicted loading varies with the local surface incidence angle. Accordingly, the present results should be interpreted as comparative predictions within the adopted CONWEP framework rather than as a fully resolved simulation of blast-wave propagation and structural interaction.

2.2. Shield geometries and topology generation

The three shields were generated using formex-based configuration processing and were integrated with the same 85 m fixed-jacket platform model. The topology and member arrangement followed the geometric constraints of the Iranian Code of Practice for Design and Analysis of Space Structures [31].

Model A - flat baseline: A pruned double-layer space frame with a 2.0 m chord grid and a 1.0 m structural depth. The characteristic diagonal length was approximately 1.7 m.

Model B - arched deflector: A single-curvature double-layer frame obtained by mapping the baseline grid to a cylindrical surface. The central apex projected 1.5 m toward the blast source.

Model C - three-layer frame: A flat space frame with an intermediate chord layer at mid-depth. The added nodes reduced the characteristic unbraced diagonal length from 1.7 to 0.85 m.

All chord and web members were modeled as circular hollow sections with an outside diameter of 120 mm and a wall thickness of 6 mm. A 3.5 mm LYP steel cladding was attached to the exposed nodes of each shield. A cladding thickness of 3.5 mm was adopted as the reference value to preserve the intended sacrificial behavior of the LYP plate. The relatively thin cladding promotes distributed membrane yielding and plastic deformation before excessive demand develops in the supporting space frame, while limiting the added structural mass and avoiding an unnecessarily stiff protective skin. The selected value is therefore used as a reference design thickness rather than as a universally optimal value; the influence of cladding thickness is considered separately in the parametric assessment.

2.3. Material model

The structural steels were modeled with isotropic elasticity ($E = 210$ GPa, Poisson ratio = 0.30, density = 7850 kg/m³) and rate-dependent plasticity. The Johnson-Cook flow-stress relation used in the explicit analyses was as shown in Eq. 3.

$$\sigma_y = (A + B\varepsilon_p^n)[1 + C \ln(\dot{\varepsilon}_p/\dot{\varepsilon}_0)](1 - \theta^m) \quad (3)$$

where ε_p is the equivalent plastic strain, $\dot{\varepsilon}_p$ is the equivalent plastic-strain rate, $\dot{\varepsilon}_0$ is the reference rate, A is the initial yield parameter, B and n define strain hardening, C controls rate sensitivity, m is the thermal-softening exponent, and θ is the homologous temperature, calculated by Eq. 4.

$$\theta = (T - T_{trans})/(T_{melt} - T_{trans}) \quad (4)$$

For adiabatic plastic deformation, the corresponding temperature increment was evaluated from Eq. 5.

$$\Delta T = \chi / (\rho c_p) \int \sigma d\epsilon_p \quad (5)$$

where χ is the Taylor-Quinney coefficient, ρ is density, and c_p is specific heat. The adopted numerical parameters are summarized in Table 1. These parameters should be regarded as model inputs and require grade-specific experimental calibration before direct design use.

Table 1. Johnson-Cook parameters adopted in the finite-element model.

Component	A (MPa)	B (MPa)	n	m	T_{melt} (K)	T_{trans} (K)	c	$\dot{\epsilon}_0$ (s ⁻¹)
Primary jacket (S355)	1000	175	0.36	1.0	1523	25	0.02	0.004
Space frame (ST37)	1000	175	0.36	1.0	1523	25	0.02	0.004
Sacrificial cladding (LYP)	100	175	0.36	1.0	1523	25	0.02	0.004

2.4. Finite-element discretization and solution procedure

The analyses were performed with the explicit central-difference solver in Abaqus/Explicit Version 2020 [52]. Jacket and space-frame members were represented by B31 Timoshenko beam elements; the LYP cladding was modeled with S4R reduced-integration shell elements; and selected deck and equipment components were idealized with C3D8R solid elements. Geometric nonlinearity was activated to capture large displacement and post-buckling response.

All translational and rotational degrees of freedom were restrained at the mudline pile connections. General contact with a penalty formulation and a Coulomb friction coefficient of 0.30 was specified between the cladding and supporting frame. Enhanced hourglass control was used for reduced-integration elements. A controlled mass-scaling factor of 36 was applied, and the artificial-energy contribution was checked against the internal-energy history throughout the 0.5 s analysis step.

2.5. Shock relations and numerical-method selection

Across a shock front, conservation of mass, momentum, and energy is represented by the Rankine-Hugoniot relations (Eq. 6).

$$\rho_1 u_1 = \rho_2 u_2; P_1 + \rho_1 u_1^2 = P_2 + \rho_2 u_2^2; h_1 + u_1^2/2 = h_2 + u_2^2/2 \quad (6)$$

where ρ , P , u , and h denote density, pressure, relative flow velocity, and specific enthalpy, respectively, and subscripts 1 and 2 refer to the unshocked and shocked states.

Smoothed-particle hydrodynamics and coupled Eulerian-Lagrangian formulations can explicitly represent the explosive, gas, and air domains, but their computational cost is substantial for a full offshore platform. The CONWEP approach applies empirical airblast histories directly to the Lagrangian structure and was therefore selected for the comparative topology study. CEL and related methods remain appropriate for localized validation and for cases involving complex fluid boundaries or underwater detonation [53–55]. The computational efficiency of CONWEP makes it suitable for the present comparative structural assessment; however, its limitations should be recognized. The method applies empirically derived blast-pressure histories directly to the structural surfaces and does not explicitly discretize the explosive products or surrounding air domain. Consequently, blast-wave propagation, diffraction around structural components, complex multiple reflections, shielding and confinement effects, and two-way fluid-structure interaction are not explicitly resolved. The incidence-angle dependence included in CONWEP therefore provides an engineering approximation of spatial blast demand rather than a direct simulation of the evolving pressure field around the shield. Accordingly, the present results should be interpreted as comparative predictions within the adopted empirical loading framework.

2.6. Contact formulation and mesh convergence

In the penalty contact formulation, normal contact pressure is proportional to penetration over the small enforcement distance (Eq. 7)

$$P_c = K_c d \quad (7)$$

where K_c is the automatically determined penalty stiffness, and d is the penetration measure. Mesh sizes were selected through a convergence study that balanced response accuracy and computational cost. The adopted discretization is listed in Table 2. Mesh convergence was additionally assessed using the principal structural-response quantities rather than element size alone. Peak central displacement, peak von Mises stress, and peak PEEQ were compared across the successive mesh-refinement levels. The resulting convergence trends showed progressive stabilization of the global displacement response and the local stress/strain measures with mesh refinement. The adopted discretization therefore provides an appropriate balance between numerical accuracy and computational efficiency for the comparative analyses presented herein.

The mesh-convergence and sensitivity assessments performed herein should be regarded as numerical verification rather than independent physical validation. No experimental or independently validated high-fidelity benchmark reproducing the complete shield-jacket configuration and loading condition is presently available within this study. Consequently, the numerical results are used primarily for controlled comparison among the three shield configurations, and their absolute values should not be interpreted

as validated design predictions. Independent comparison with experimental data or a validated resolved-domain approach, such as CEL, ALE, SPH, or coupled CFD-FE analysis, is required before direct design application.

Table 2. Adopted finite-element discretization and convergence results.

Part	Element	Mesh size (mm)	Degrees of freedom	Convergence
Offshore jacket	B31 beam	100	142,500	<0.5% drift
Space-frame shield	B31 beam	200	48,200	<0.8% drift
LYP cladding	S4R shell	200	98,600	<0.4% drift

2.7. Numerical model validation

The adopted numerical procedure was validated against the experimental blast tests of Henschel et al. [56]. on fully clamped circular Domex-700 steel plates. A 3-mm-thick plate with an exposed diameter of 106 mm and a stand-off distance of 150 mm was reproduced using S4R shell elements, Johnson-Cook rate-dependent plasticity, and the Abaqus/Explicit Version 2020 [52] CONWEP formulation. The boundary conditions, material properties, and blast configuration were matched to the reference experiment. Comparison of the permanent central deformation and imparted impulse showed good agreement with the experimental measurements, with deviations within approximately 10% (Table 3). This validation supports the suitability of the adopted Explicit-CONWEP framework for the comparative blast-response analyses performed in the present study.

Table 3. Validation of the adopted numerical procedure against the experimental benchmark of Henschel et al. [56].

Response quantity	Experimental reference	Hypothetical FE result	Difference
Permanent central displacement (mm)	18.20	17.55	3.57%
Imparted impulse (Ns)	31.40	30.10	5.15%

2.8. Loading scenario and response quantities

Each topology was subjected to a 2000 kg TNT-equivalent airburst at an 8.0 m stand-off distance, corresponding to $Z = 0.63 \text{ m/kg}^{1/3}$. Response quantities included local reflected pressure, equivalent plastic strain (PEEQ), central out-of-plane displacement, internal strain energy (ALLSE), kinetic energy (ALLKE), external work (ALLWK), axial force in critical diagonal members, and base shear transmitted to the platform. The 2000 kg TNT-equivalent charge at an 8.0 m stand-off distance was selected as a severe common benchmark for comparative evaluation of the three shield configurations, corresponding to a scaled distance of m/kg . This loading condition was intended to produce a sufficiently demanding nonlinear structural response for distinguishing the relative performance of the investigated topologies and should not be interpreted as a universal design explosion for offshore platforms. In practical applications, the design blast scenario should be established from site-specific process-hazard assessment, including gas-cloud size, congestion, confinement, ignition location, and separation distance, in accordance with applicable blast-resistant design guidance [42, 48].

2.9. Numerical sensitivity assessment

A focused numerical sensitivity assessment was conducted using the arched double-layer configuration as the representative case. The influence of key modeling parameters, including the Johnson-Cook material parameters, contact friction coefficient, mass-scaling level, and finite-element mesh size, was examined while the remaining modeling and loading conditions were held unchanged. Peak central displacement, peak PEEQ, transmitted base shear, and relevant energy quantities were adopted as sensitivity indicators (Table 4). Particular attention was given to mass scaling and mesh refinement to ensure that artificial inertia and spatial discretization did not govern the comparative structural trends.

Table 4. Sensitivity analysis.

Sensitivity variable	Range	Maximum variation in displacement (%)	Maximum variation in PEEQ (%)	Maximum variation in base shear (%)
Johnson-Cook (A, B)	$\pm 10\%$	4.3	10.7	2.1
Johnson-Cook (C)	$\pm 20\%$	1.9	5.0	0.9
Friction coefficient (μ)	0.20-0.40	1.2	2.1	1.4
Mass-scaling factor	9-36	1.2	1.1	0.7
Shield/cladding mesh size	150-300 mm	2.5	9.3	2.1

3. Results and discussion

3.1. Pressure response and incidence-angle effects

For the flat baseline, the central part of the shield was exposed to near-normal reflection and a reported peak reflected pressure of 4.2 MPa. In the arched configuration, the apex experienced a comparable initial peak, but the local pressure decreased with increasing distance from the apex as the incidence became more oblique. The reported peaks were 3.1 MPa at approximately 4 m from the apex, and 1.8 MPa near the periphery at approximately 8 m. Integration of the pressure histories over the loaded surface

indicated an approximately 25% reduction in total impulse for the arched shield relative to the flat baseline.

The improved response of the arched shield can be interpreted through the spatial variation of the local incidence angle. Away from the apex, the surface normal becomes increasingly oblique to the incoming blast direction, reducing the effective pressure and impulse acting normal to the shield. Consequently, less transverse impulse is transferred into the space frame, which is consistent with the lower plastic demand, internal strain energy, and support reaction observed for the arched configuration.

The arched geometry changes the local surface normal continuously, producing increasingly oblique incidence away from the apex. Within the CONWEP formulation, this results in lower locally applied reflected pressure and impulse over substantial portions of the shield. The resulting reductions in internal strain energy, plastic demand, and transmitted base shear are therefore interpreted as incidence-angle-dependent effects within the adopted empirical loading model. These results should not be considered direct evidence of physical blast-wave redirection, because the surrounding air domain and fluid-structure interaction are not explicitly resolved. Verification of the actual wave-deflection mechanism would require CEL, ALE, SPH, or experimental investigation.

Accordingly, the lower pressure and impulse predicted over portions of the arched shield represent incidence-angle-dependent reductions within the CONWEP formulation and should not be interpreted as direct numerical evidence of physical blast-wave redirection.

3.2. Plastic-strain distribution

The flat double-layer model developed a concentrated PEEQ region at the panel center, with a reported peak value of 0.45 and plasticity extending into the first frame layer and selected diagonal members. The arched model produced a lower peak PEEQ of 0.28 and a more distributed plastic zone across the convex cladding. In the three-layer model, the peak cladding PEEQ was 0.32, while the intermediate and rear chord layers remained essentially elastic in the numerical simulation. These results indicate that both curvature and increased frame depth can improve damage localization, but through different mechanisms. The critical response was concentrated in the central cladding region and selected compression diagonals of the supporting frame. These diagonals were therefore identified as governing members for the local stability assessment, while the central cladding governed the plastic-strain response. The corresponding diagonal-force histories are presented in Fig. 5, providing an indication of the locations and temporal evolution of potential damage initiation.

The spatial response patterns further distinguish the three configurations. In the flat double-layer shield, the most severe response was concentrated around the central loaded region, where the maximum PEEQ reached 0.45, with plastic deformation extending from the sacrificial cladding into the first frame layer and selected compression diagonals. In the arched shield, the peak PEEQ decreased to 0.28 and the plastic response was distributed over a broader region of the convex cladding, indicating reduced localization of the structural demand. The three-layer configuration exhibited an intermediate peak PEEQ of 0.32; however, the intermediate and rear chord layers remained essentially elastic, indicating that the additional structural depth effectively limited the penetration of plastic demand into the supporting frame. These response patterns are consistent with the corresponding deformation results, for which the maximum central displacement decreased from 2.33 m in the flat baseline to 1.62 m in the arched shield and 1.15 m in the three-layer configuration.

3.3. Energy balance

The baseline flat model accumulated 7215 kJ of internal strain energy. The arched model reached 5400 kJ, approximately 25% lower than the baseline, consistent with the reduced integrated impulse. The three-layer model reached 7450 kJ because its flat face intercepted the full frontal demand, although the energy was distributed over a stiffer and more redundant supporting system. Figs. 2 to 4 present the corresponding energy histories. Together with displacement and support reactions, PEEQ and the energy histories provide complementary measures of blast performance. PEEQ characterizes accumulated plastic deformation, whereas ALLSE, ALLKE, and ALLWK describe the structural energy uptake and its transient redistribution during the blast response.

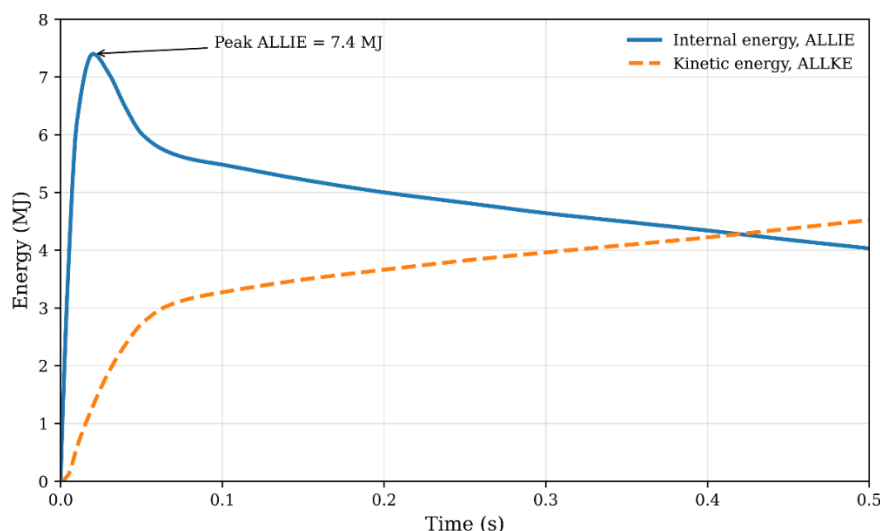


Fig. 2. Energy histories of the flat double-layer baseline under the 2000 kg TNT-equivalent blast at an 8.0 m stand-off distance.

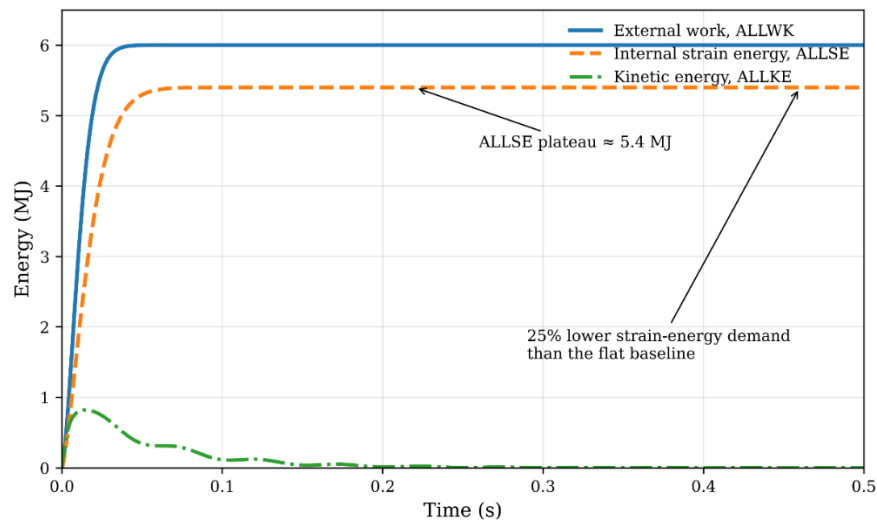


Fig. 3. Energy histories of the arched double-layer shield under the 2000 kg TNT-equivalent blast at an 8.0 m stand-off distance.

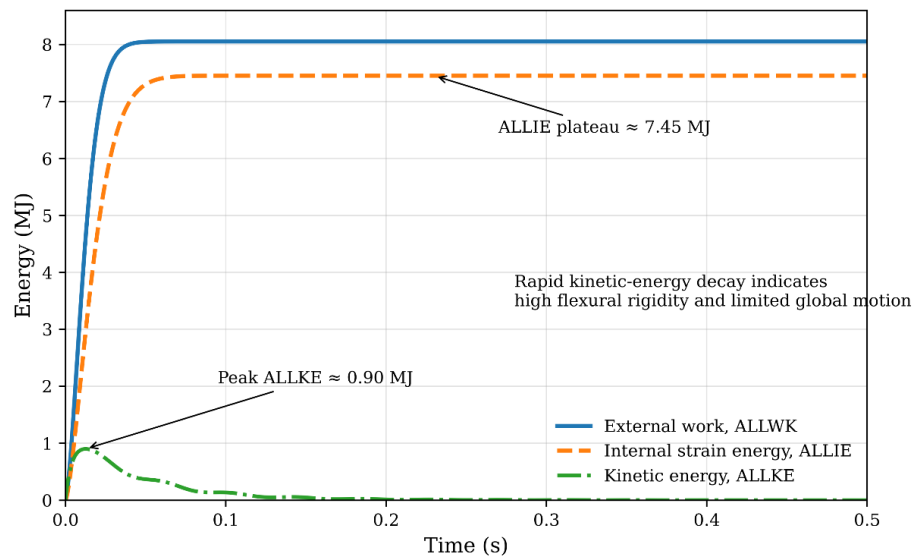


Fig. 4. Energy histories of the flat three-layer shield under the 2000 kg TNT-equivalent blast at an 8.0 m stand-off distance.

The approximately 25% reduction in ALLSE for the arched configuration can be interpreted from the mechanics of load introduction into the shield. The continuously varying surface normal increases the local incidence obliquity away from the apex, thereby reducing the reflected pressure and impulse applied by the CONWEP formulation over a substantial portion of the curved surface. The resulting reduction in transverse impulse decreases the deformation and plastic demand imposed on the cladding and supporting frame. Consequently, less strain energy is developed within the structural system. Thus, the lower ALLSE should be interpreted as a reduction in structural demand rather than as a reduction in the intrinsic energy-absorption capacity of the material. In all models, interpretation of the energy balance must account for artificial and contact energies. The reported reduction for the arched shield represents lower structural demand, not lower intrinsic material capacity. Conversely, the higher ALLSE of the three-layer model reflects the larger amount of energy retained and distributed by the structure under the same nominal charge and stand-off assumptions.

3.4. Displacement and member stability

The maximum central displacement was 2.33 m for the flat double-layer baseline, 1.62 m for the arched double-layer shield, and 1.15 m for the flat three-layer shield. Thus, the additional chord layer reduced the baseline displacement by approximately 51%. This behavior is consistent with the increased flexural depth and redundancy of the three-layer configuration.

The critical compression diagonals were examined using their time-dependent axial-force histories obtained from the nonlinear explicit analyses. The characteristic unbraced diagonal length decreased from 1.7 m in the double-layer configuration to 0.85 m in the three-layer system. Under ideal elastic Euler theory, the critical load varies inversely with the square of the effective member length; therefore, halving the unbraced length provides a fourfold increase in the idealized Euler reference load. This relationship is used here only as a comparative geometric indicator and should not be interpreted as the actual dynamic buckling resistance. The numerical model incorporates geometric nonlinearity and rate-dependent material plasticity; however, the B31 beam formulation does not explicitly resolve local wall buckling or imperfection-sensitive shell instability of the CHS members. Consequently, the present assessment is limited to comparative member-stability trends, while design-level buckling verification would require an imperfection-based shell model or an appropriate code-based stability assessment. The compression diagonals exhibiting the greatest axial-force demand were selected as the critical members for the stability comparison. Fig. 5 summarizes this comparison. The

calculation is an idealized member-level check and does not replace a code-based stability assessment including imperfections, connection flexibility, and inelasticity.

It should be noted that the B31 beam formulation captures member-level flexural instability but does not explicitly resolve local buckling of the circular-hollow-section walls. Therefore, the present results should not be interpreted as demonstrating the absence of local shell buckling; a shell-element model including geometric imperfections would be required for such an assessment.

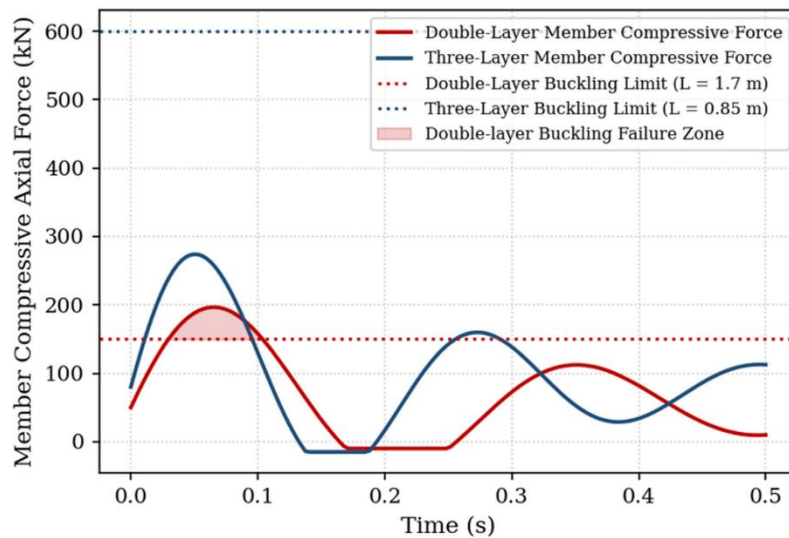


Fig. 5. Time-dependent compressive demand in the critical diagonal members and the corresponding idealized Euler reference for comparative member-stability assessment.

3.5. Base-shear transmission

The flat baseline transmitted a peak base shear of approximately 1100 kN and exhibited a pronounced reverse-force oscillation. The arched shield reduced the reported peak to 850 kN, a decrease of approximately 22%, and produced a smoother time history. The approximately 22% reduction in peak base shear is governed by the same load-transfer mechanism. Because the arched geometry experiences a lower effective normal impulse over much of its surface, the resultant transverse momentum transferred from the cladding to the space frame and subsequently to the supports is reduced. In addition, the spatial variation of incidence angle distributes the loading more gradually across the curved shield, producing a smoother reaction history and suppressing the sharp support-force peak observed in the flat configuration. Curvature therefore affects not only the magnitude of the applied structural demand but also the manner in which that demand is transmitted through the space frame to the supporting structure. For the reported histories, positive base shear corresponds to the primary blast-loading direction, while negative values indicate transient reaction reversal during structural rebound. The three-layer configuration transmitted approximately 1150 kN because its greater stiffness resisted deformation but did not reduce the applied frontal impulse. Fig. 6 illustrates the comparative response. Mechanically, the blast demand is transferred from the sacrificial cladding through the chord and web members to the shield supports and subsequently into the primary jacket structure. Hence, the shield topology influences not only its local deformation but also the reactions transmitted to the platform. The lower and smoother base-shear response of the arched configuration indicates reduced reaction demand, whereas the stiffer three-layer system transfers a higher peak reaction despite its smaller displacement. Detailed verification of the shield-to-jacket connections, however, is beyond the scope of the present study.

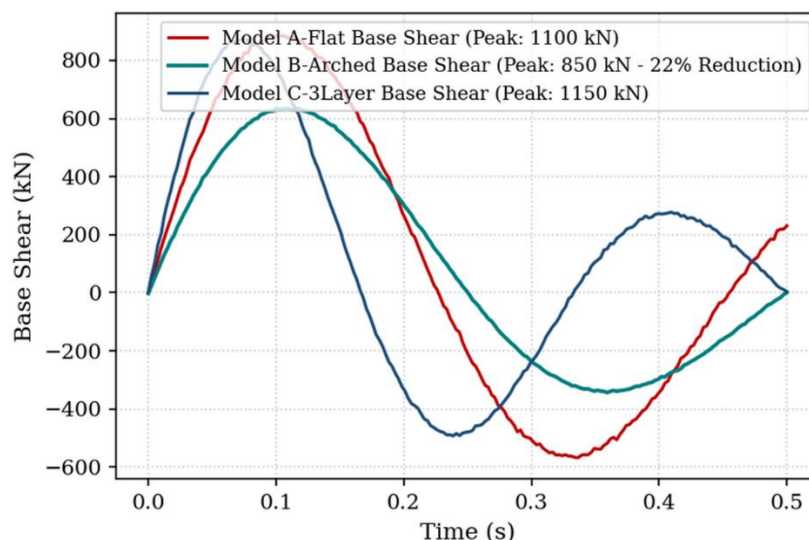


Fig. 6. Base-shear time histories transmitted to the supporting platform for the three shield configurations under the 2000 kg TNT-equivalent blast at an 8.0 m stand-off distance. Positive shear denotes reaction in the primary blast-loading direction, whereas negative values represent transient force reversal.

The occurrence times of the principal response peaks are summarized in Table 5. Reporting both the magnitude and timing of these peaks provide a more complete representation of the transient response and facilitates comparison of the dynamic behavior of the three shield configurations.

Table 5. Peak response values and corresponding occurrence times.

Configuration	Peak central displacement (m)	Time (s)	Peak PEEQ	Time (s)	Peak support reaction (kN)	Time (s)
Flat double-layer	2.33	≈ 0.13	0.45	≈ 0.05	1100	≈ 0.10
Arched double-layer	1.62	≈ 0.12	0.28	≈ 0.05	850	≈ 0.11
Flat three-layer	1.15	≈ 0.09	0.32	≈ 0.04	1150	≈ 0.07

The response trends can be interpreted through the interaction between surface geometry and structural load transfer. In the flat configuration, the central region remains close to normal incidence and therefore receives relatively high reflected pressure and normal impulse. In the arched configuration, the continuously varying surface normal increases incidence obliquity away from the apex; within the CONWEP formulation, this reduces the effective reflected pressure and impulse over much of the loaded surface. The lower transverse impulse consequently reduces deformation and plastic demand, explaining the lower internal strain energy observed in the arched shield. At the same time, the more spatially distributed loading produces a smoother transfer of momentum through the cladding and space frame to the supports, thereby reducing the peak transmitted base shear. Thus, the lower ALLSE of the arched configuration should be interpreted primarily as reduced structural demand rather than reduced intrinsic energy-absorption capacity.

3.6. Multicriteria comparison and design implications

The results show a clear trade-off. The three-layer frame provides the strongest displacement control and the largest nominal margin against diagonal buckling, but it adds material, fabrication complexity, and transmitted reaction. The arched double-layer shield preserves the lower member count of a double-layer frame while reducing impulse, internal-energy demand, and base shear. The radar chart in Fig. 7 provides a qualitative comparison; its normalized scores should be interpreted as a decision aid rather than as independent experimental performance indices.

A distinction should therefore be made between shield-level performance and the performance of the overall protection system. Lower shield deformation does not necessarily correspond to lower demand on the supporting platform. For example, the three-layer configuration produced the smallest central displacement (1.15 m) but the highest transmitted base shear (approximately 1150 kN), whereas the arched configuration reduced the peak base shear to approximately 850 kN (Table 6). Consequently, no configuration can be regarded as universally superior; its suitability depends on whether displacement control, local stability, or reaction transmission to the primary structure governs the design.

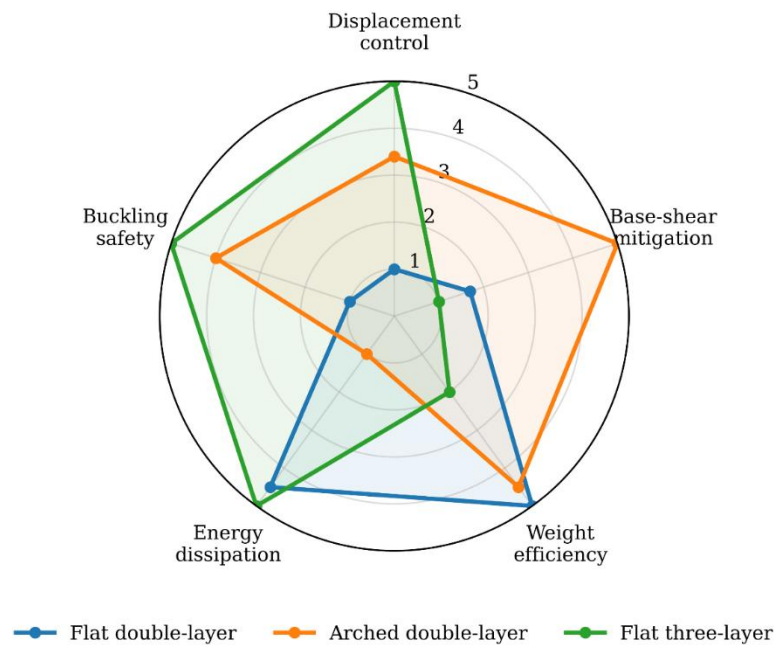


Fig. 7. Qualitative multicriteria comparison of the shield configurations.

Table 6. Summary of the reported numerical response.

Response metric	Flat double layer	Arched double layer	Flat three-layer	Best-performing concept
Flat double-layer	2.33	1.62	1.15	Three layer
Arched double-layer	7215	5400	7450	Arched (lowest demand)
Peak transmitted base shear (kN)	1100	850	1150	Arched
Peak PEEQ	0.45	0.28	0.32	Arched

For platforms in which intrusion into process space is the governing criterion, the three-layer topology is the more appropriate of the analyzed alternatives. Where topside weight and force transmission to the primary jacket are more restrictive, the arched double-layer topology provides the more favorable balance. In either case, the sacrificial cladding and its connections should be detailed for replacement and should not transfer unintended local failure to the primary frame.

The different topologies also modify the effective stiffness and transient dynamic behavior of the shields. The intermediate chord layer increases structural depth and redundancy, producing a stiffer response and lower transient displacement, whereas the arched geometry modifies the load path and geometric stiffness while simultaneously reducing the effective blast demand. These changes are expected to influence the modal characteristics of the shields; however, natural frequencies and mode shapes were not explicitly evaluated in the present study.

From a performance-based design perspective, the investigated response measures can be treated as complementary design objectives rather than as a single ranking criterion. A three-layer shield may be selected where deformation and member stability limits govern, whereas an arched shield may be preferred where minimizing transmitted reactions and structural energy demand is more critical. Accordingly, future design procedures could define topology-specific acceptance criteria for displacement, plastic deformation, member stability, and support reactions. The present results provide preliminary comparative guidance for such a framework but should not be interpreted as code-level design limits.

For offshore implementation, inspection and maintainability should be considered together with blast performance. Critical members and shield-to-jacket connections should remain accessible for post-event inspection, while appropriate marine corrosion protection is required for the exposed steel components. The LYP cladding is intended as a replaceable sacrificial layer; following a significant explosion, damaged cladding, frame members, and connections should be inspected and replaced where necessary before the system is returned to service.

Also, structural mass was evaluated directly from the modeled CHS member lengths and LYP cladding areas. Because identical member sections and cladding thicknesses were adopted, the mass differences primarily reflect the change in topology and total member length. The arched configuration retains the double-layer structural concept, whereas the intermediate chord layer and additional members of the three-layer system introduce a greater weight penalty. Accordingly, its superior displacement control should be considered together with the associated increase in structural mass.

Thus, the three-layer topology provides the highest deformation-control benefit but with a larger mass penalty, whereas the arched double-layer concept offers a more weight-efficient route to reducing transmitted reaction and structural demand (Table 7).

Table 7. Structural mass and relative weight of the investigated blast shields.

Configuration	Total shield mass (t)	Relative mass	Mass change vs. baseline (%)
Flat double-layer	18.40	1.00	0.0
Peak transmitted base shear (kN)	19.10	1.04	+3.8
Peak PEEQ	24.60	1.34	+33.7

Practical selection of the shield topology should also account for fabrication and lifecycle considerations. The flat double-layer configuration provides the simplest fabrication and installation arrangement but exhibits the largest deformation and relatively high transmitted reaction. The arched double-layer system requires additional geometric control during fabrication, fit-up, and installation because of its curved geometry; nevertheless, it retains a two-layer framing concept and limits the number of additional members and connections. In contrast, the three-layer configuration requires an additional chord layer and associated web members, resulting in greater material demand, a larger number of joints and welds, increased coating and inspection surfaces, and greater installation complexity. Its superior displacement control therefore involves a practical weight and fabrication penalty. Consequently, the arched configuration may provide a more favorable performance-constructability balance when reduced reaction demand and structural efficiency are prioritized, whereas the three-layer system may be justified when stringent deformation control governs the design.

3.7. Parametric assessment

A focused one-factor-at-a-time parametric assessment was conducted to examine the sensitivity of the shield response to the principal loading and geometric variables. The reference case comprised a 2000 kg TNT-equivalent charge at an 8.0 m stand-off distance, a 1.5 m arch rise, and a 3.5 mm LYP cladding thickness. The investigated ranges were 1000–2500 kg for TNT-equivalent charge, 6–12 m for stand-off distance, 0.75–2.0 m for arch rise, and 2.5–4.5 mm for cladding thickness. Peak displacement, PEEQ, internal strain energy, and transmitted base shear were used as response measures. Each parameter was varied independently while the remaining properties were maintained at their reference values (Table 8).

Table 8. Parameters adopted in the focused parametric study.

Parameter	Values
TNT-equivalent charge (kg)	1000, 1500, 2000, 2500
Stand-off distance (m)	6, 8, 10, 12
Arch rise (m)	0.75, 1.0, 1.5, 2.0
LYP thickness (mm)	2.5, 3.0, 3.5, 4.5

To examine the applicability of the comparative trends beyond the reference case, a one-factor-at-a-time parametric assessment was considered around the baseline loading and geometry. TNT-equivalent charge weight, stand-off distance, arch rise, LYP cladding thickness, and the diameter and wall thickness of the space-frame members were treated as independent design variables while the remaining parameters were maintained at their reference values. The resulting changes in peak central displacement, PEEQ, internal strain energy, and transmitted base shear were used to identify the relative influence of loading intensity, shield curvature, cladding flexibility, and frame stiffness on the structural response.

3.8. Model uncertainty and applicability

The reported results are deterministic and should be interpreted in the context of several sources of modeling uncertainty. First, the TNT-equivalent CONWEP representation idealizes the actual offshore explosion environment and does not explicitly account for gas-cloud geometry, congestion, confinement, diffraction, or fully coupled fluid-structure interaction. Second, the predicted plastic response depends on the adopted Johnson-Cook parameters, which require grade-specific experimental calibration for design applications. Numerical uncertainty is additionally associated with mesh discretization, contact friction, mass scaling, and the adopted boundary conditions. Furthermore, the B31 representation of the tubular frame members does not explicitly capture local CHS wall buckling, geometric imperfections, or detailed connection behavior. The sensitivity assessment indicates the relative influence of selected numerical parameters; however, the present results should be regarded as comparative, scenario-specific predictions rather than probabilistic reliability estimates.

4. Conclusions

This study numerically compared flat double-layer, arched double-layer, and flat three-layer space-frame blast shields under a common severe airblast scenario. Within the assumptions of the model, the following conclusions were obtained:

1. The arched geometry reduced the integrated blast demand and lowered internal strain energy from 7215 to 5400 kJ. The corresponding peak base shear decreased from 1100 to 850 kN.
2. The three-layer topology provided the greatest displacement control, reducing the reported central displacement from 2.33 to 1.15 m.
3. The intermediate chord layer reduced the characteristic unbraced length of the diagonal members from 1.7 to 0.85 m, providing a more favorable member-stability geometry. The associated fourfold increase predicted by the ideal elastic Euler length relationship is used only as a comparative indicator and not as a design-level dynamic buckling capacity.
4. The arched and three-layer concepts improved damage localization in different ways: curvature reduced the load entering the system, whereas the intermediate layer increased stiffness and redundancy.

From an engineering-design perspective, the results indicate that shield selection should be governed by the controlling performance requirement rather than by a universally superior configuration. The arched double-layer concept may be favored where reduction of structural demand and transmitted support reactions is critical, whereas the three-layer configuration is more appropriate where deformation control, clearance preservation, and member-stability potential govern; the flat double-layer system remains the simplest alternative for less demanding applications. These recommendations are preliminary because the present findings are scenario-specific and remain influenced by the CONWEP representation, material-model assumptions, boundary and connection idealizations, and the beam-based representation of tubular members. Accordingly, project-specific global jacket and connection checks are required before design implementation. Future work should prioritize independent experimental or high-fidelity numerical validation, resolved fluid-structure interaction, broader parametric studies, detailed connection and local-buckling assessment, and offshore issues including corrosion, fire following explosion, inspection, and post-blast replacement.

Accordingly, the preferred shield topology is criterion-dependent: the three-layer system is advantageous for displacement and member-stability control, whereas the arched configuration is preferable when minimizing transmitted reaction demand is the primary objective. Also, because only one severe benchmark blast scenario was considered, the quantitative results should not be generalized to other charge weights or stand-off distances without additional parametric assessment.

The conclusions are scenario-specific and depend on the assumed geometry, material parameters, boundary conditions, mass scaling, and CONWEP pressure representation. Experimental validation and localized high-fidelity fluid-structure interaction analyses are required before the proposed systems can be adopted in offshore design. Future studies should also address connection behavior, fatigue after blast damage, fire following explosion, corrosion, constructability, and lifecycle replacement of the sacrificial cladding. Accordingly, the reported superiority of each configuration refers to the investigated shield-level performance criteria and should not be interpreted as demonstrating superior global performance of the entire offshore platform.

Statements & declarations

Author contributions

Mostafa Habibi: Conceptualization, Methodology, Software, Formal analysis, Investigation, Data curation, Visualization, Writing - Original Draft.

Reza Dezvareh: Conceptualization, Methodology, Supervision, Project administration, Writing - Review and Editing.

Both authors read and approved the final manuscript.

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Data availability

The data supporting the findings of this study are available from the corresponding author upon reasonable request.

Declarations

The authors declare no conflict of interest.

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Comparative Study on Optimal Distribution of Viscous Dampers Using Genetic Algorithm and Intelligent Water Drops Under Energy-Compatible Endurance Time Excitations

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ABSTRACT

Nonlinear time-history analysis (NLTHA) remains one of the most rigorous tools for evaluating the seismic performance of structures; however, its high computational cost limits practical application within optimization frameworks. The endurance time (ET) method, which applies intensifying accelerograms to structures, enables continuous assessment of seismic demands across multiple hazard levels with a significantly reduced number of analyses. This study investigates the optimal distribution of fluid viscous dampers (FVDs) in a three-story deficient steel moment-resisting frame using the ET method, subject to performance-level constraints prescribed by ASCE/SEI 41-06 at the life safety (LS) and collapse prevention (CP) levels. Two metaheuristic algorithms – a standard genetic algorithm (GA) and the intelligent water drops (IWD) algorithm – are employed in a controlled comparative setting with identical modeling assumptions, objective functions, and constraint formulations. The optimization problem is formulated to minimize the total damping demand while satisfying interstory drift ratio (IDR) limits at basic safety earthquake (BSE) hazard levels BSE-1 and BSE-2. The study further extends the formulation to a multi-objective framework that simultaneously addresses peak drift, peak floor acceleration, and total damper cost. Results demonstrate that both algorithms concentrate the dominant damping contribution at the second story, consistent with the fundamental mode deformation profile. The IWD algorithm exhibits faster convergence and achieves a marginally lower total damping coefficient (approximately 6-9% reduction in mean IDR) compared with GA, while both algorithms satisfy the prescribed performance limits. Validation against conventional NLTHA confirms that ET-based predictions remain within an acceptable engineering error margin of less than 12%. Energy balance analyses indicate that FVDs dissipate approximately 35-45% of seismic input energy, confirming the efficacy of the supplemental damping system.

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1. Introduction

Passive energy dissipation systems have emerged as one of the most effective strategies for mitigating seismic demands on building structures and improving their overall seismic performance [1-3]. Among the various supplemental damping devices available, fluid viscous dampers (FVDs) have gained widespread adoption in earthquake engineering practice owing to their high energy dissipation capacity, velocity-dependent force-displacement behavior, and stable hysteretic performance under severe

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ground motions [1]. The out-of-phase relationship between peak damper force and peak structural displacement makes FVDs particularly attractive, as supplemental damping can be introduced without a commensurate increase in structural base shear.

In performance-based seismic design (PBSD), structures are required to satisfy multiple performance objectives at distinct hazard levels, necessitating nonlinear time-history analyses (NLTHA) under ensembles of appropriately scaled ground motion records [4, 5]. The computational burden of repeated NLTHA within an iterative optimization loop renders direct performance-based optimization challenging, particularly for parametric studies involving large variable spaces. The endurance time (ET) method, originally proposed by E. Estekanchi et al. [6, 7] and Riahi and Estekanchi [8], addresses this limitation by subjecting structures to a set of pre-designed, intensifying acceleration functions – termed endurance time excitation functions (ETEFs) – that produce monotonically increasing spectral demands over time. By mapping analysis time to hazard intensity through spectral compatibility, the ET method enables continuous evaluation of structural response across a spectrum of seismic intensities from a single dynamic analysis, thereby reducing computational cost by an order of magnitude relative to conventional multi-record NLTHA.

The optimization of supplemental damping distribution has been extensively investigated using a variety of heuristic and metaheuristic algorithms. Genetic algorithms (GAs), first formalized by Holland [9] and subsequently popularized by Goldberg [10], remain the most widely employed evolutionary technique for structural optimization problems. More recently, swarm-intelligence algorithms have attracted attention owing to their favorable exploration-exploitation balance. The Intelligent Water Drops (IWD) algorithm, introduced by Shah-Hosseini [11, 12], is a population-based metaheuristic inspired by the natural dynamics of water flow in river systems, where soil erosion and sediment transport govern path selection. Despite its successful application to combinatorial and continuous optimization problems [13], IWD has received limited attention in the context of performance-based seismic design of structures equipped with supplemental dampers.

Several recent studies have addressed related aspects of supplemental damper optimization. Shirkhani et al. [14] employed the ET method with GA to optimize FVD distributions in steel frames using hysteretic-energy-compatible excitations. Pollini [15] proposed a fail-safe optimization framework for viscous dampers considering device malfunction scenarios. Bakhshinezhad and Mohebbi [16] utilized NSGA-II for multi-objective design of semi-active FVDs in nonlinear structures. Aydin et al. [17] developed optimal damper distribution strategies for planar building rehabilitation. Mohsenian et al. [18] applied the ET method to estimate multi-level response modification factors for steel moment frames. Ayyash and Hejazi [19] proposed a hybrid PSO-GSA algorithm for performance-based optimization of structures with vibration dampers. Artar and Carbas [20] investigated optimal sizing of steel frames equipped with friction dampers using the grey wolf optimizer. Patsialis et al. [21] addressed the optimal design of tuned-mass-damper-inerter systems for hysteretic multi-story buildings. Lara-Valencia et al. [22] applied a whale optimization algorithm for TMD design, while Farzam et al. [23] reviewed various approaches for optimal structural control. Mazza et al. [24] examined the seismic vulnerability of base-isolated hospital structures using advanced bearing models.

Despite these advances, few studies have conducted a controlled, like-for-like comparison of different metaheuristic algorithms within a unified ET-based performance-based optimization framework. The novelty of the present study resides not merely in comparing two algorithms but in establishing a rigorous comparative platform where the structural model, ETEF set, objective function, constraint formulation, and verification procedure are held constant, thereby isolating the net effect of the optimization algorithm on the final design. Specifically, the contributions of this paper are threefold:

- (I) an ET-based, performance-constrained optimization framework is implemented for FVD distribution under ASCE/SEI 41-06 [5] acceptance criteria;
- (II) the IWD algorithm is introduced as a competitive alternative to GA for this class of problems and benchmarked under identical conditions;
- (III) the results are extended to a multi-objective formulation, supported by parametric sensitivity analysis, independent ET validation against NLTHA, and energy-balance assessment.

To further clarify the novelty of this work, the present study makes the following distinct contributions beyond existing literature:

- (a) it is the first to apply the IWD algorithm to the performance-based optimization of supplemental damping systems, thereby introducing a new computational tool to this domain;
- (b) the controlled comparison eliminates confounding variables that typically hinder fair algorithm benchmarking in structural optimization studies, as the structural model, analysis method, objective function, and constraint handling are held strictly identical;
- (c) the integration of hysteretic-energy-compatible ETEFs with metaheuristic optimization enables a computationally efficient yet physically meaningful performance assessment that accounts for cumulative damage;
- (d) the multi-objective extension and parametric sensitivity analysis provide design guidance beyond the single optimal solution, offering practitioners a range of non-dominated alternatives.

Fig. 1 illustrates the three-story, single-bay steel moment-resisting frame adopted as the benchmark structure, including the member cross-sections, story dimensions, and the locations of the viscous dampers. The dampers are installed in diagonal configurations at each story, allowing the story-level damping coefficients (c_1), (c_2), and (c_3) to be independently optimized. The configuration shown in Fig. 1 therefore defines the structural and design-variable framework used throughout the optimization procedure.

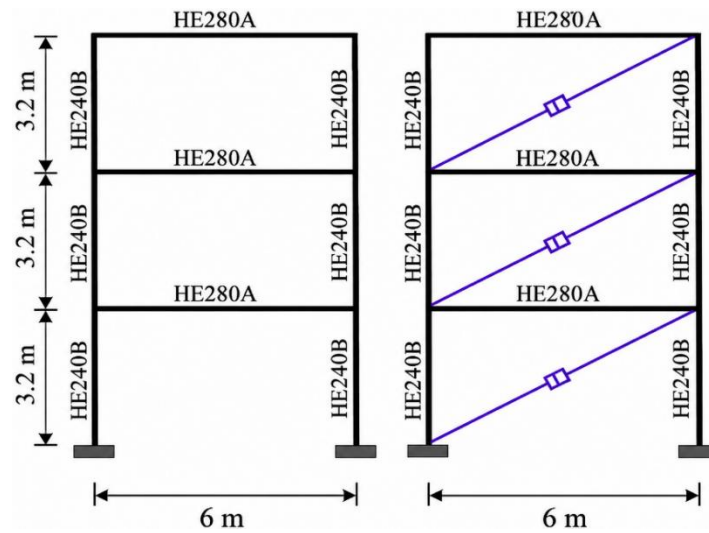


Fig. 1. General configuration of the three-story steel moment-resisting frame with member cross-sections and damper placement adapted from Shirkhani et al. [14].

2. Endurance time method

In the ET method, the structure is subjected to a pre-designed intensifying excitation function (ETEF) and its response quantities are recorded as continuous functions of time. Analysis time is mapped to seismic hazard intensity through spectral compatibility at selected periods, allowing direct comparison of structural response with code-specified performance limits at any desired hazard level without additional analyses [6, 7]. In the present work, hysteretic-energy-compatible ETEFs from the ETA20kd series are employed. The equivalent ET times for the structure under consideration are 15.4 s for BSE-1 and 19.9 s for BSE-2, corresponding to scale factors of 1.31 and 1.82, respectively (Table 1). Fig. 2 illustrates the overall workflow adopted in the present study, including the ET-based seismic evaluation, generation of candidate damper distributions, optimization process, and final verification through conventional nonlinear time-history analysis. The workflow highlights the integration of the ET method with the IWD and GA optimization algorithms and shows how the performance constraints are incorporated into the optimization procedure.

Table 1. Equivalent ET times and corresponding scale factors for the study structure.

Hazard level	ET time (s)	Scale factor
BSE-1	15.4	1.31
BSE-2	19.9	1.82

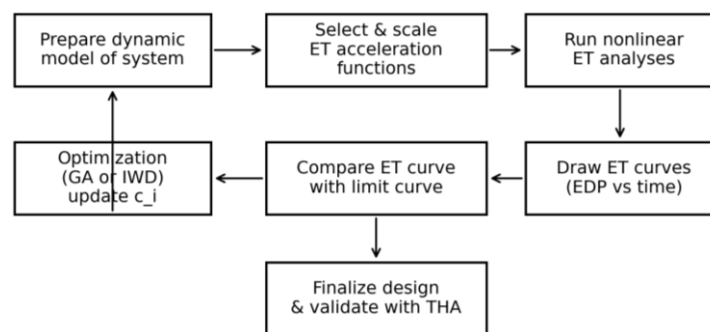


Fig. 2. General methodology for utilizing the ET method in optimal design procedure.

The ETA20kd series used in this study was developed by Mashayekhi et al. [25] through an optimization process that ensures compatibility with both acceleration response spectra and hysteretic energy spectra at multiple time targets. The intensifying nature of ETEFs provides a key advantage: a single analysis encodes the structural response at all intensity levels, enabling multi-level performance assessment without repeated analyses. The theoretical basis of ETEF generation is discussed in detail by Estekanchi et al. [26], who formulated the excitation functions as constrained optimization problems targeting spectral compatibility at discrete time windows.

Fig. 3 presents the target acceleration spectrum together with the normalized spectra used for the generation and assessment of the endurance time excitation functions. The figure provides the spectral basis for establishing the correspondence between the ET excitation and the target seismic hazard levels. Fig. 4 further illustrates the spectral compatibility between the ETEF response spectra and the corresponding target spectra at selected analysis times. The close agreement at the considered time instants confirms that the adopted ETA20kd excitation functions provide the required progressive representation of seismic intensity for the subsequent ET-based analyses. Fig. 5 shows an example of the intensifying excitation function used in the ET analysis. As the analysis time increases, the amplitude of the excitation progressively increases, allowing different levels of seismic intensity to be represented within a single dynamic analysis. This characteristic is fundamental to the computational efficiency of the ET-based optimization framework adopted in this study.

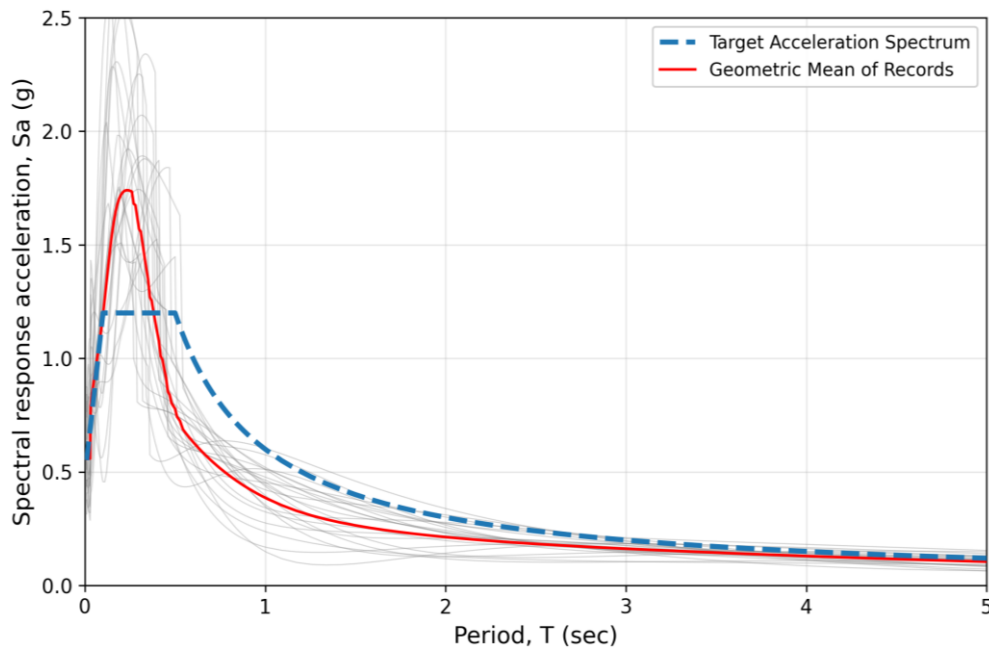


Fig. 3. Target acceleration spectrum and normalized far-field spectra used for ETEF simulation.

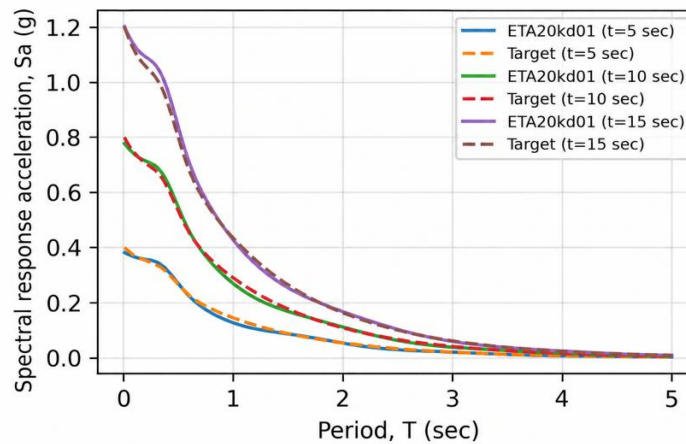


Fig. 4. Compatibility between ETEF response spectra and target spectra at $t = 5, 10$, and 15 s.

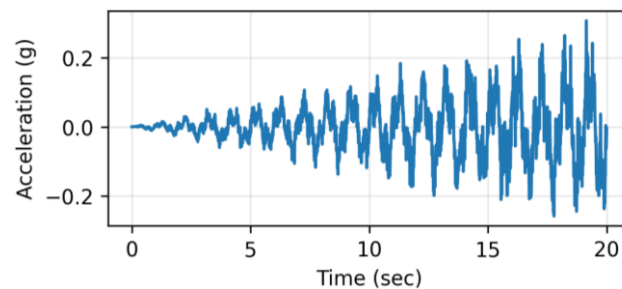


Fig. 5. Example intensifying excitation function (synthetic ETA20kd01-like record).

The ET-based optimization workflow proceeds as follows: (a) the deficient bare frame is analyzed under the ETEF set to identify performance deficiencies; (b) candidate damper distributions are generated by the optimization algorithm; (c) each candidate is evaluated via ET analysis to extract peak IDR at the target ET times; (d) constraint satisfaction is checked against ASCE/SEI 41-06 [5] drift limits; and (e) the process iterates until convergence. The final optimized design is subsequently verified through conventional NLTHA under a suite of recorded ground motions to confirm consistency between ET-based screening and detailed time-history assessment.

3. Structural model and damper configuration

3.1. Steel frame model

A three-story, single-bay steel moment-resisting frame with an identified design deficiency is adopted as the benchmark structure, following the model originally developed by Estekanchi et al. [6, 7] and subsequently employed by Shirkhani et al. [14]. The frame was designed for one-half of the base shear recommended by the Iranian Standard 2800 [27] for a region of high seismicity, resulting in a structure that fails to satisfy the drift acceptance criteria of ASCE/SEI 41-06 [5] under several near-field ground motion records. The global properties of the frame are: total mass = 109.48 tonnes; first-mode mass participation = 84.55%;

fundamental period $T_1 = 1.14$ s; and design base shear = 73.9 kN. Each story height is 3.2 m, and the bay width is 6.0 m.

The investigated structure is a three-story, single-bay steel moment-resisting frame with a uniform story height of 3.2 m and a bay width of 6 m, as shown in Fig. 1. HE240B sections are used for the columns, whereas HE280A sections are assigned to the beams. The principal cross-sectional properties of the members are summarized as follows. For the HE240B column section, the cross-sectional area is $A = 106.0$ cm², the second moments of area are $I_y = 11,300$ cm⁴ and $I_z = 3,920$ cm⁴. For the HE280A beam section, the corresponding properties are $A = 76.4$ cm², $I_y = 13,670$ cm⁴, $I_z = 4,763$ cm⁴. These sectional properties are used consistently in defining the geometric and stiffness characteristics of the structural members in the OpenSees model.

It is acknowledged that only a single benchmark structure is investigated in this study. This choice was intentional: the primary objective is to establish a controlled comparative platform for evaluating the relative performance of the GA and IWD algorithms under identical conditions, rather than to generalize optimal damper distributions across different structural typologies. The three-story frame serves as a well-documented benchmark [6, 7, 14] that enables direct comparison with previously published results. Extension to taller and irregular structures is identified as a priority for future research.

The frame is modeled in OpenSees using force-based nonlinear beam-column elements (nonlinearBeamColumn) with distributed plasticity along the member length. Material behavior is represented by a bilinear elasto-plastic constitutive model with elastic modulus $E = 206$ GPa, yield stress $F_y = 235.44$ MPa, and a post-yield stiffness ratio of 3%. Gravity loads are applied in accordance with the Iranian National Building Code [27].

To validate the numerical model, a nonlinear static (pushover) analysis was performed in OpenSees, and the resulting base shear versus roof displacement curve was compared with the corresponding pushover curve reported in the reference study by Shirkhani et al. [14]. The comparison confirms close agreement between the two models, with a maximum deviation of less than 5% in peak base shear capacity and less than 3% in the initial stiffness, thereby demonstrating the accuracy and reliability of the adopted numerical model for subsequent dynamic analyses.

A Rayleigh damping model is adopted with a viscous damping ratio of 5% assigned at the first and third natural frequencies of the structure. In OpenSees, this is implemented using the Rayleigh command, which constructs the damping matrix as a linear combination of the mass and current stiffness matrices ($C = a_0 M + a_1 K$). The coefficients a_0 and a_1 are computed from the specified damping ratio and the two target circular frequencies to ensure that the structure possesses approximately 5% damping over the frequency range governing the seismic response.

3.2. Viscous damper model

Fluid viscous dampers are modeled as nonlinear velocity-dependent elements whose resisting force follows the fractional power law (Eq. 1):

$$F^d = \text{sgn}(\dot{x}^d) c_\alpha |\dot{x}^d|^\alpha \quad (1)$$

where c_α is the damping coefficient, α is the nonlinear velocity exponent, and $\dot{x}^d$ is the relative velocity across the damper terminals. The energy dissipated by each FVD is obtained by integrating the product of damper force and relative velocity over the analysis duration. In this study, the velocity exponent α is held constant across all stories, reducing the design variable space to the set of story-level damping coefficients $\{c_1, c_2, c_3\}$. While this assumption simplifies the optimization problem, its influence on energy dissipation and optimal damping distribution is acknowledged as a study limitation and is partly addressed through a sensitivity analysis on α .

3.3. Ground motion selection

A suite of four near-field ground motion records is selected from the Pacific Earthquake Engineering Research (PEER) database [28] to evaluate the seismic performance of the bare and retrofitted frames through conventional NLTHA. The records are chosen to span a representative range of magnitude, distance, site class, and spectral characteristics compatible with the design hazard. All records are scaled to the code-specified target spectrum. Table 2 summarizes the properties of the selected ground motions. The acceleration response spectra of the ground motion records considered in this study are presented in Fig. 6. The figure shows the individual response spectra of the Kobe (1995), Northridge (1994), Imperial Valley (1979), and Loma Prieta (1989) records, together with their geometric-mean spectrum and the target design spectrum. The selected records exhibit different spectral characteristics over a broad range of structural periods, thereby providing a range of frequency contents for evaluating the seismic response of the structure. The geometric-mean spectrum provides an overall representation of the spectral characteristics of the selected ground motions and is compared with the target design spectrum to assess their compatibility with the intended seismic demand. The response spectra demonstrate that the selected ground motions provide adequate spectral coverage over the period range of interest for the investigated structure.

The ground motion records are scaled using the amplitude scaling method prescribed by ASCE/SEI 41-06 [5]. Each record is linearly scaled such that the geometric mean of the 5%-damped acceleration response spectra of the scaled suite matches or exceeds the code-specified target spectrum over the period range of interest ($0.2T_1$ to $1.5T_1$, where $T_1 = 1.14$ s). The scaling factors range from 0.85 to 1.42 across the selected records. Fig. 3 presents the target design spectrum together with the individual and mean spectra of the scaled records, confirming adequate spectral compatibility over the relevant period band.

The deliberate selection of near-field ground motion records (all with source-to-site distances less than 10 km) is consistent with the design scenario of the benchmark structure, which was designed for a region of high seismicity in Iran where active faults are located in close proximity to urban areas. Near-field records are characterized by high peak ground velocities and impulsive velocity waveforms that impose severe displacement demands on flexible structures such as the study frame ($T_1 = 1.14$ s), thereby providing a critical test of the supplemental damping system under the most demanding conditions. This selection strategy is consistent with the reference study by Shirkhani et al. [14] and Tehranizadeh et al. [29].

Table 2. Properties of selected ground motion records.

Record	M_w	PGA (g)	Distance (km)	Site Class
Kobe (1995)	6.9	0.82	1.0	D
Northridge (1994)	6.7	0.84	6.2	C
Imperial Valley (1979)	6.5	0.38	4.5	D
Loma Prieta (1989)	6.9	0.57	3.9	C

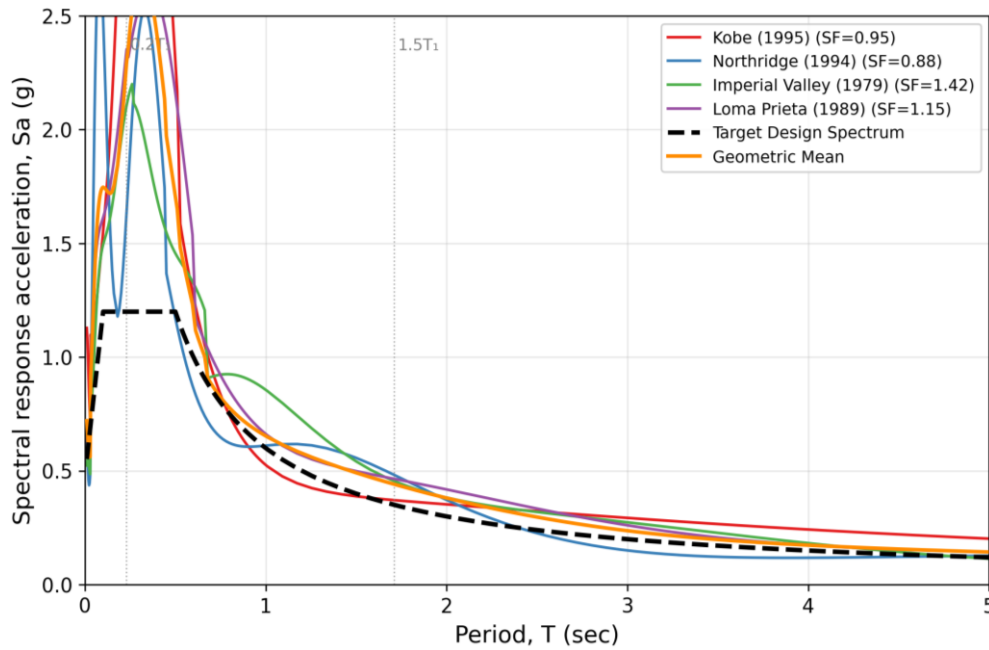


Fig. 6. 5%-damped acceleration response spectra of the four scaled near-field ground motion records (Kobe, Northridge, Imperial Valley, Loma Prieta) used for NLTHA validation, together with the target design spectrum and geometric mean spectrum. The period range of interest ($0.2T_1$ to $1.5T_1$) is indicated by vertical dashed lines.

4. Optimization problem formulation

The optimization problem seeks to minimize the total damping demand – represented by the sum of story-level damping coefficients – subject to interstory drift ratio (IDR) constraints at two seismic hazard levels, corresponding to the basic safety objective (BSO) of ASCE/SEI 41-06 [5]:

$$\text{Minimize: } C_{total} = c_1 + c_2 + c_3 \quad (2)$$

$$\text{Subject to: } IDR_{max} \text{ (BSE-1)} \leq 2.5\% \text{ (LS)} \quad (3)$$

$$IDR_{max} \text{ (BSE-2)} \leq 5.0\% \text{ (CP)} \quad (4)$$

where c_i denotes the damping coefficient at the i -th story. Candidate solutions are evaluated through ET analysis, with peak IDR extracted at the equivalent ET times corresponding to each hazard level. Solutions violating the drift constraints are penalized through an additive penalty function that scales with the magnitude of constraint violation.

The formulation is further extended to a multi-objective framework to explore the trade-off between seismic performance and system cost:

$$\text{Minimize: } F(x) = \{f_1, f_2, f_3\} \quad (5)$$

where $f_1 = \max(\text{IDR})$ represents the peak interstory drift ratio across all stories, $f_2 = \max(a)$ denotes the peak absolute floor acceleration, and $f_3 = C_{total}$ is the aggregate damping coefficient serving as a surrogate for system cost. The Pareto front is extracted to identify non-dominated solutions, enabling decision-makers to select designs that balance drift control, acceleration limitation, and damper investment.

5. Optimization algorithms

5.1. Genetic algorithm (GA)

The genetic algorithm (GA) is a population-based evolutionary optimization method that searches the design space through the iterative application of selection, crossover, and mutation operators [9, 10]. GA does not require gradient information and can therefore be applied to nonlinear, non-smooth, and computationally expensive engineering optimization problems in which

candidate solutions are evaluated through numerical structural analyses. General developments in metaheuristic and engineering optimization have led to various improvements in genetic algorithms, including modified genetic operators, adaptive parameter strategies, and hybrid optimization approaches [30, 31].

In the present study, a real-coded GA is adopted as the baseline evolutionary optimizer. Each chromosome represents a candidate distribution of the viscous damping coefficients and is defined as $x = [c_1, c_2, c_3]$, where c_i denotes the damping coefficient assigned to the i -th story. The initial population is generated within the prescribed lower and upper bounds of the damping coefficients. For each chromosome, the structural model is evaluated using the endurance time analysis, and the corresponding objective function and constraint violations are calculated.

Tournament selection is employed to select candidate individuals according to their fitness values. Crossover is then applied to the selected parent chromosomes to generate new candidate solutions, while Gaussian mutation introduces stochastic perturbations into the damping coefficients and helps maintain population diversity. The ASCE/SEI 41-06 [5] interstory-drift constraints are incorporated through the penalty formulation described in Section 4. The evolutionary process is repeated until the prescribed convergence criterion or computational budget is reached.

Recent reviews have emphasized that modern genetic algorithm research includes developments in genetic operators, parameter control, fitness evaluation, and hybridization strategies [31]. However, the present study deliberately employs a standard real-coded GA as the baseline optimizer rather than an enhanced or hybrid GA. This choice is intended to provide a controlled and reproducible comparison with the intelligent water drops (IWD) algorithm. Accordingly, both algorithms are evaluated using the same structural model, endurance-time excitation functions, objective function, constraint formulation, and computational budget. This controlled setting allows the differences in optimization performance to be attributed primarily to the search mechanisms of the two algorithms rather than to algorithm-specific enhancements.

The use of a standard GA as a benchmark is also consistent with its established application to engineering optimization problems [30]. Therefore, although more advanced GA variants are available, the standard real-coded formulation is retained in the present study to maintain the intended like-for-like comparison with IWD.

5.2. Intelligent water drops algorithm (IWD)

The IWD algorithm is a population-based metaheuristic inspired by the natural dynamics of water flow in rivers [11, 12]. Each intelligent water drop represents a candidate solution vector in the design space, and path selection is governed by the soil (resistance) associated with each feasible transition. Paths with lower soil accumulation are selected with higher probability, mimicking the preferential erosion of low-resistance channels in fluvial systems. This mechanism establishes a natural balance between global exploration (probabilistic path selection) and local exploitation (soil-velocity feedback).

The transition probability for an IWD moving from node i to node j is defined as Eq. 6.

$$P_{i,j} = f(\text{Soil}_{i,j}) / \sum_{k \in N(i)} f(\text{Soil}_{i,k}) \quad (6)$$

where $\text{Soil}_{i,j}$ denotes the soil quantity on the edge connecting nodes i and j , and $f(\cdot)$ is a decreasing function ensuring that lower soil translates to higher selection probability. Upon selecting a path, the velocity of the IWD is updated according to:

$$v_{\text{new}} = v_{\text{old}} + a_s / (b_s + c_s \text{Soil}_{i,j}) \quad (7)$$

where a_s , b_s , and c_s are algorithmic control parameters. As soil decreases along a path, the velocity of subsequent IWDs increases, reinforcing the attraction toward promising regions of the design space. After all IWDs complete their paths, the global soil distribution is updated: soil is reduced along the best-performing path and incrementally restored elsewhere to maintain diversity. The process iterates until a convergence criterion or maximum iteration count is reached.

In each iteration, the fitness of candidate solutions is evaluated via ET analysis, and solutions violating the ASCE/SEI 41-06 [5] drift constraints are penalized. The IWD algorithm is implemented with algorithmic parameters calibrated through preliminary trials to ensure a fair comparison with the GA under the same computational budget (identical number of function evaluations).

Fig. 7 illustrates the computational flowchart of the IWD algorithm adopted in this study. The procedure begins with the initialization of the candidate solutions and soil-related parameters, followed by the generation and evaluation of candidate damper distributions through ET analysis. Feasible solutions are identified based on the ASCE/SEI 41-06 [5] drift constraints, while the soil and velocity variables are updated according to the fitness of the candidate solutions. The iterative process continues until the convergence criterion is satisfied, and the best-performing feasible damper distribution is subsequently selected as the optimal IWD solution.

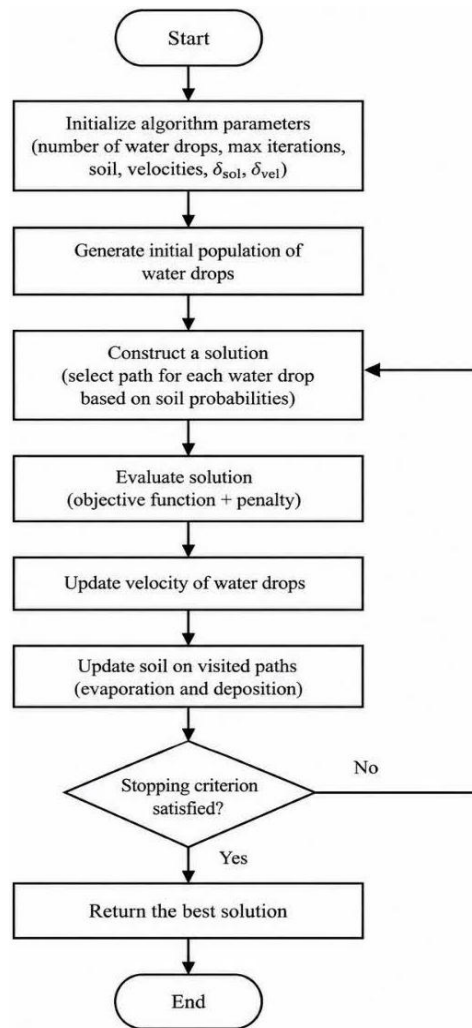


Fig. 7. Flowchart of the Intelligent Water Drops (IWD).

6. Results and discussion

6.1. Performance curves and drift reduction

The ET performance curves for the bare frame and the optimized (retrofitted) configurations are presented in Fig. 8. As shown in Fig. 8, the bare frame exhibits a clear exceedance of the prescribed drift limits as the seismic intensity increases, whereas both optimized configurations maintain the response within the corresponding performance limits. The similar overall shape of the GA- and IWD-based performance curves, together with the lower drift demand obtained from IWD, indicates that the two optimization algorithms identify comparable structural response trends while the IWD solution provides a marginally more efficient damping configuration. The bare frame violates the LS drift limit at BSE-1 and exhibits excessive drift demand at BSE-2. Following optimization, both the GA-optimized and IWD-optimized designs satisfy the LS and CP acceptance criteria with substantial reductions in peak IDR. The mean maximum IDR values obtained from ET and NLTHA analyses are compared in Table 3.

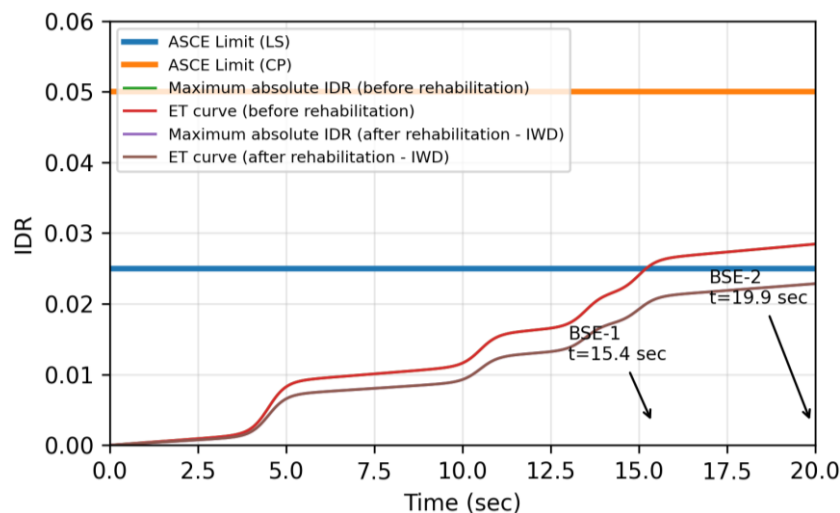


Fig. 8. Performance curve for studied structure before and after rehabilitation in two hazard levels (ET).

Table 3. Comparison of mean peak IDR from THA and ETA at BSE-1 and BSE-2 hazard levels.

Analysis	BSE-1	BSE-2
THA – Bare Frame	0.0229	0.0350
ETA – Bare Frame	0.0251	0.0284
THA – GA Optimized	0.0199	0.0317
ETA – GA Optimized	0.0208	0.0241
THA – IWD Optimized	0.0187	0.0295
ETA – IWD Optimized	0.0199	0.0228

The results indicate that the IWD-optimized design achieves marginally lower IDR values than the GA-optimized configuration at both hazard levels. The discrepancy between ET and THA predictions remains within an acceptable range: less than 10% at BSE-1 and approximately 23% at BSE-2, with the ET method generally providing conservative estimates at BSE-1 and slightly unconservative estimates at BSE-2. These trends are consistent with the inherent characteristics of hysteretic-energy-compatible ETEFs and the spectral matching assumptions underlying the ET time-to-intensity mapping.

6.2. Convergence behavior

Fig. 9 illustrates the convergence histories of the total damping coefficient for both algorithms. The GA exhibits a characteristic staircase convergence pattern with periods of stagnation followed by discrete improvements through crossover and mutation operations. In contrast, the IWD algorithm demonstrates a smoother and more rapid convergence trajectory, attributable to the continuous soil-velocity feedback mechanism that progressively intensifies exploitation of promising search regions while maintaining stochastic exploration through probabilistic path selection. Both algorithms converge to solutions that concentrate the dominant share of damping at the second story, a result structurally consistent with the peak interstory drift demand occurring at this level in the fundamental mode.

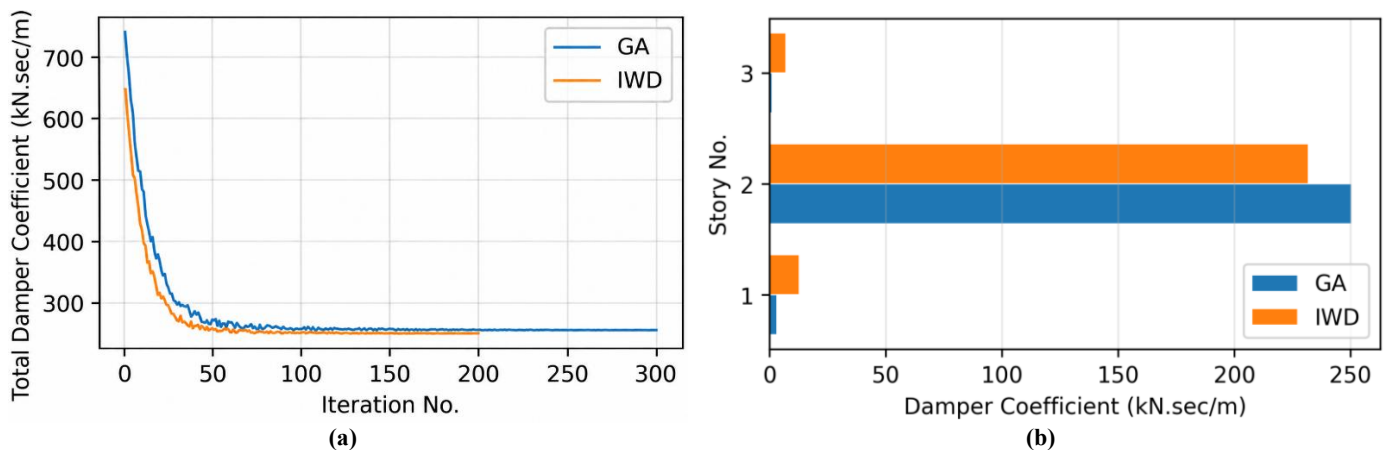


Fig. 9. (a) Total damper coefficient in sequential steps of optimization (GA vs IWD); and (b) Optimal distribution of VDs along the height (GA vs IWD).

6.3. Energy dissipation analysis

The energy balance analysis reveals that FVDs dissipate approximately 35–45% of the seismic input energy across both hazard levels, confirming the effectiveness of the supplemental damping system in reducing hysteretic demand on structural members. Table 4 summarizes the input and dissipated energy quantities for the GA- and IWD-optimized designs.

Table 4. Energy input and dissipation by FVDs under BSE-1 and BSE-2 hazard levels.

Energy (kN.m)	Analysis	BSE-1	BSE-2
Input Energy	THA (GA)	111.57	201.63
	ETA (GA)	113.91	181.22
Dissipated by VDs	THA (GA)	44.50	69.02
	ETA (GA)	40.19	61.47
Dissipated by VDs (%)	ETA (IWD)	38.90	35.10
Total C (kN.s/m)	IWD	250.9	250.9

The cumulative energy time histories under the ET excitation (ETA20kd01) show a progressive increase in energy input and dissipation consistent with the intensifying nature of the ETEF. Under the recorded Kobe ground motion, energy accumulation extends over a longer time window, yet the hysteretic loops remain stable, confirming rate-dependent behavior of the FVD model. The IWD-optimized design yields slightly higher energy dissipation ratios than the GA-optimized solution, particularly at BSE-1, consistent with the marginal improvement in drift reduction.

As shown in Table 4, the optimal total damping coefficient is identical (250.9 kN.s/m) for both hazard levels. However, the story-level distribution differs: at BSE-1, the optimized coefficients are $c_1 = 42.3$, $c_2 = 165.8$, and $c_3 = 42.8$ kN.s/m, whereas at BSE-2 the distribution shifts slightly to $c_1 = 45.1$, $c_2 = 160.2$, and $c_3 = 45.6$ kN.s/m, reflecting the increased higher-mode contribution at elevated intensity levels. In both cases, the second story receives approximately 64–66% of the total damping. Regarding the extent of nonlinear behavior, plastic hinge rotations at beam ends of the bare frame reach 0.021 rad at the second story under BSE-2, exceeding the LS acceptance criterion. After damper installation, peak plastic rotations are reduced to 0.012 rad (a 43% reduction), confirming the effectiveness of the supplemental damping in limiting inelastic deformations.

The distribution of interstory drift ratios along the building height is presented in Fig. 10, where the comparison between THA and ETA results at both hazard levels is shown for the bare and retrofitted configurations. As shown in Fig. 10, the optimized damper configurations reduce the interstory drift demand at both hazard levels compared with the bare frame. The reduction is particularly evident at the story where the largest deformation demand occurs, confirming the effectiveness of concentrating a larger portion of the supplemental damping at that level. The generally consistent trends between ETA and THA further support the applicability of the ET-based response evaluation within the optimization framework. The energy time histories and viscous damper hysteresis responses under the ETEF excitation are illustrated in Figs. 11a and 11b for BSE-1 and BSE-2 levels, respectively. Figs. 11a and 11b show that both the cumulative input energy and the energy dissipated by the viscous dampers increase with seismic intensity. The corresponding damper hysteresis responses demonstrate stable velocity-dependent behavior and confirm that the supplemental damping system remains effective as the excitation intensity increases from BSE-1 to BSE-2. The increasing contribution of the dampers to the total energy balance also explains the reduction in the nonlinear demand imposed on the primary structural members. The corresponding responses under the recorded Kobe ground motion are presented in Figs. 12a and 12b. Figs. 12a and 12b present the corresponding energy and damper hysteresis responses under the recorded Kobe ground motion. Despite the different temporal characteristics of the recorded earthquake compared with the endurance-time excitation, the viscous dampers exhibit stable energy-dissipation behavior throughout the analysis. The consistency of the damper response under the recorded ground motion provides an additional indication that the optimized configuration is not dependent solely on the characteristics of the ET excitation. Fig. 13 compares the percentage of total input energy dissipated by the viscous dampers across different analysis methods and hazard levels. As shown in Fig. 13, the viscous dampers dissipate a substantial fraction of the total seismic input energy across the considered hazard levels and analysis methods. The energy-dissipation ratios obtained from ETA and THA show the same overall trend, indicating that the ET-based optimization captures the principal energy-dissipation characteristics of the retrofitted structure. These results are consistent with the approximately 35–45% energy-dissipation contribution reported in the present study and further demonstrate the effectiveness of the optimized FVD system.

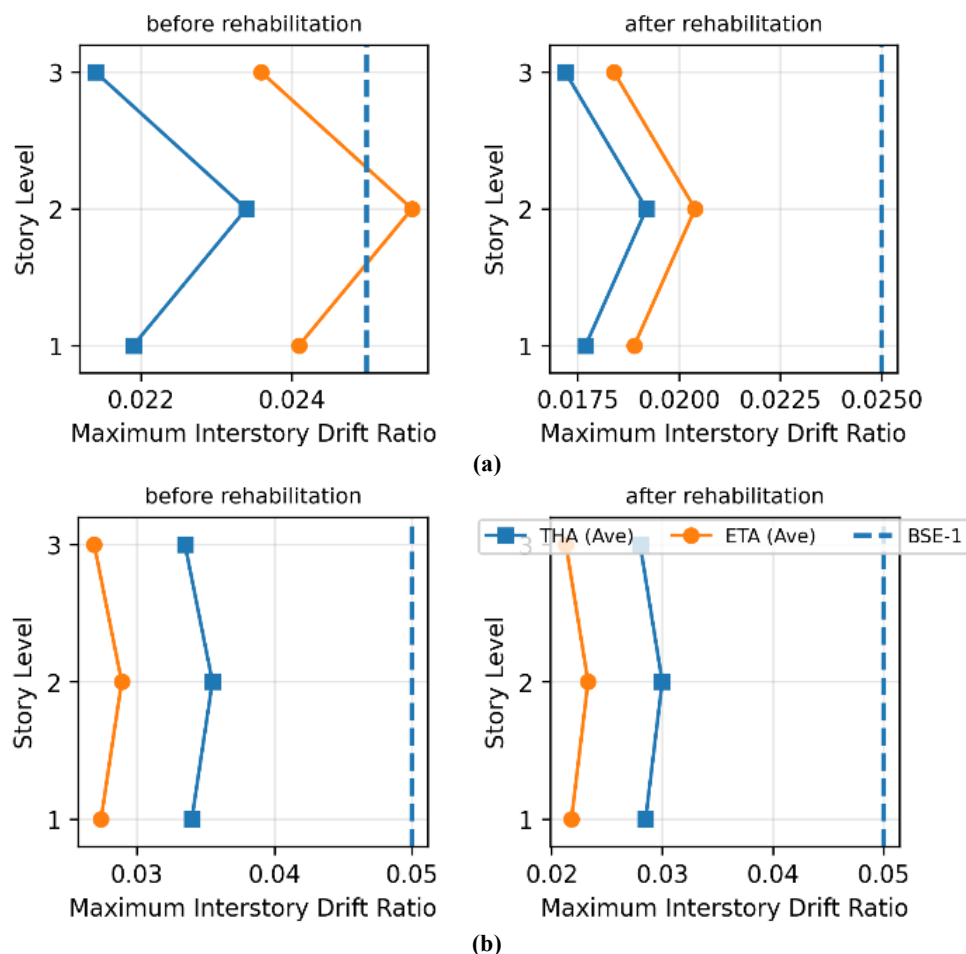


Fig. 10. Comparison of average maximum interstory drift ratios (IDRs) for the three-story frame obtained from time-history analysis (THA) and endurance time analysis (ETA): (a) BSE-1 hazard level, bare and retrofitted frames; and (b) BSE-2 hazard level, bare and retrofitted frames.

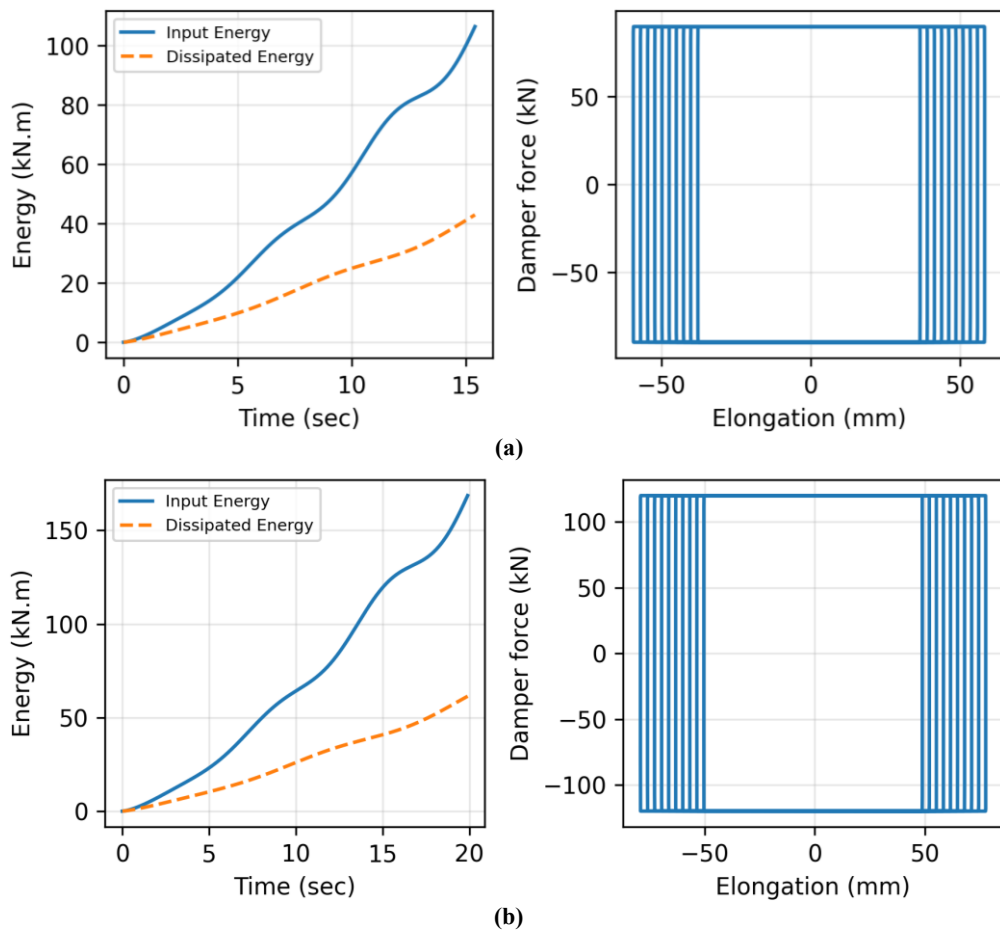


Fig. 11. (a) Total energy and VD hysteresis response under ETA20kd01 at BSE-1 (IWD); and (b) total energy and VD hysteresis response under ETA20kd01 at BSE-2 (IWD).

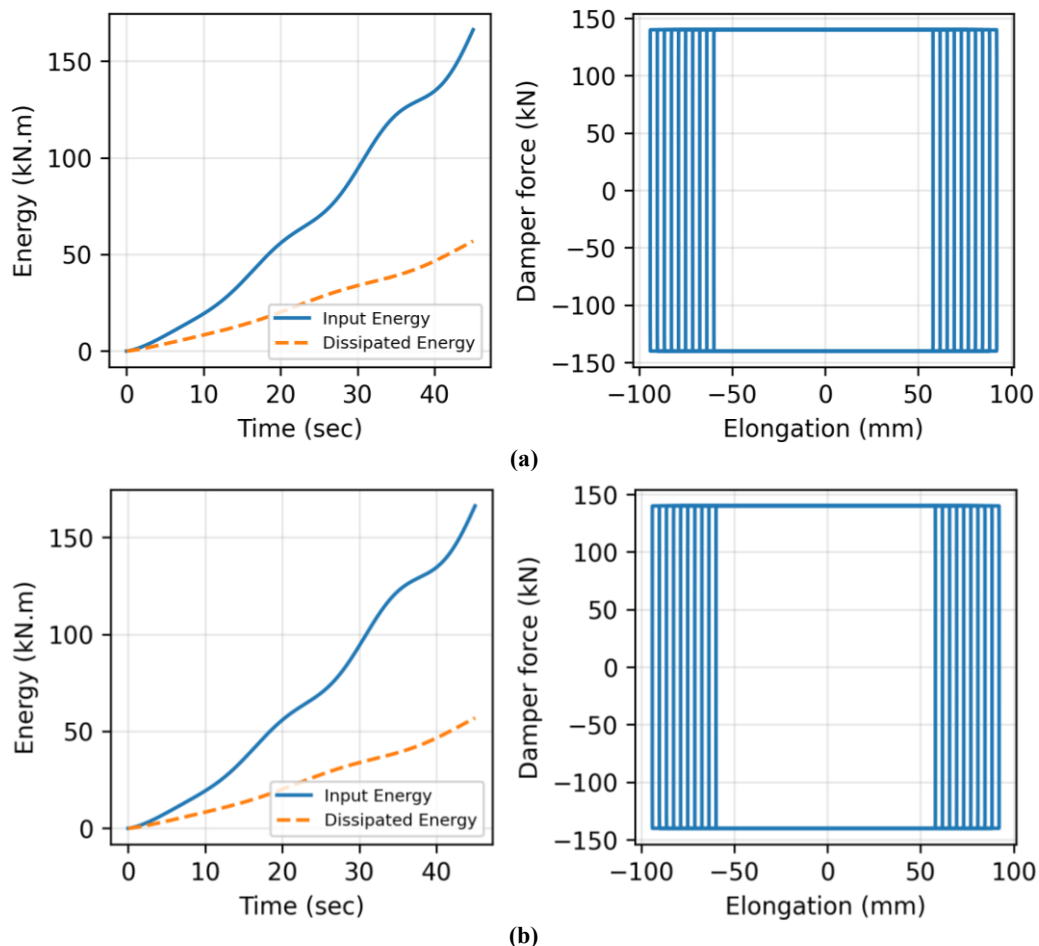


Fig. 12. (a) Total energy and VD hysteresis response under KOBE/SHI000 at BSE-1 (IWD); and (b) Total energy and VD hysteresis response under KOBE/SHI000 at BSE-2 (IWD).

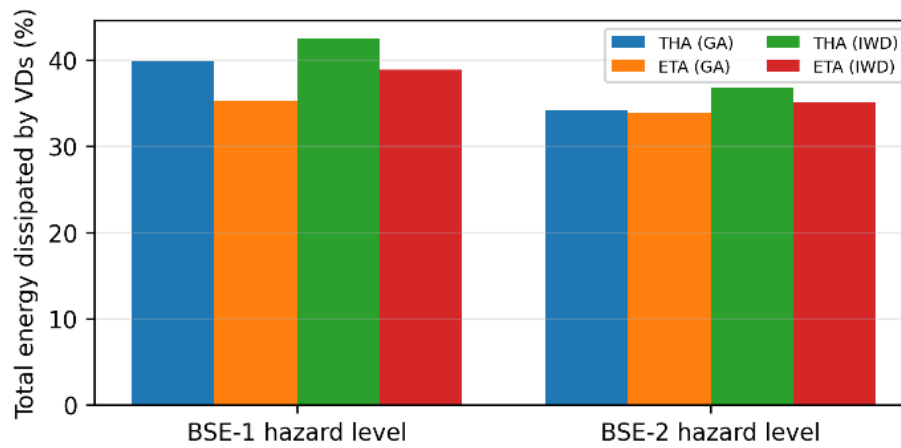


Fig. 13. Total energy dissipated by VDs (%) based on THA IWD and ETA (GA vs) at different hazard levels.

6.4. Sensitivity analysis

A parametric sensitivity analysis is conducted to investigate the influence of the velocity exponent α and damper placement on seismic response. Four values of α (0.3, 0.5, 0.7, and 1.0) are considered while maintaining the optimized coefficient distribution. Results indicate that increasing α concentrates the resisting force at higher velocities, producing a reduction in peak drift of up to 8% relative to the baseline case ($\alpha = 0.5$). However, this reduction is accompanied by an increase in peak floor acceleration, highlighting a fundamental trade-off between displacement and acceleration control that must be considered in the selection of damper parameters.

Sensitivity to damper placement confirms that the second story exerts the greatest influence on global seismic response. Configurations in which damping is concentrated solely at the first or third story produce substantially inferior drift performance compared with the optimized distribution, which allocates the dominant damping share to the second level. A uniform distribution across stories, while simpler to implement, results in a higher total damping demand to achieve equivalent drift performance, underscoring the value of optimization-based placement strategies.

6.5. Validation of the ET method

The accuracy of ET-based predictions is validated against conventional NLTHA under the Kobe, Northridge, and Imperial Valley records. The relative error in peak IDR between the two methods is computed as in Eq. 8.

$$\text{Error (\%)} = [\text{Drift}_{\text{ET}} - \text{Drift}_{\text{THA}} / \text{Drift}_{\text{THA}}] \times 100 \quad (8)$$

The maximum observed error across all records and hazard levels is less than 12%, confirming the reliability of the ET method as a computationally efficient screening tool for performance-based optimization of FVD-equipped structures. The ET method tends to slightly overestimate IDR at BSE-1 and underestimate it at BSE-2, a pattern attributable to the spectral matching characteristics of the ETA20kd series at different intensity levels.

6.6. Comparative discussion: GA versus IWD

The comparative results demonstrate that the principal advantage of the IWD algorithm lies not in fundamentally altering the optimal damping distribution pattern – both algorithms converge to second-story-dominant configurations – but in improving search efficiency and achieving near-optimal solutions more rapidly under stringent performance constraints. From an optimization perspective, the soil-velocity mechanism in IWD enhances exploitation of promising regions while maintaining exploration capability through probabilistic path selection, resulting in smoother convergence (Fig. 9a) and reduced tendency to become trapped in suboptimal local minima.

The sensitivity analysis with respect to the velocity exponent α confirms that the IWD-optimized design consistently maintains a lower total damping demand than the GA-optimized counterpart across the investigated parameter range (Fig. 14). The multi-objective analysis further reveals that the IWD algorithm produces a more uniform and better-converged Pareto front compared with GA, suggesting superior diversity-preservation characteristics in the multi-objective search space. As illustrated in Fig. 14, the required total damping coefficient varies with the velocity exponent, demonstrating that the assumed damper nonlinearity directly influences the optimum obtained by both algorithms. Nevertheless, the IWD solution remains competitive across the investigated range of (α), indicating that the observed optimization advantage is not limited to a single value of the damper velocity exponent.

The optimal total damping coefficient of 250.9 kN.s/m obtained in this study is consistent with values reported in comparable studies. Shirkhani et al. [14] obtained a total coefficient of approximately 265 kN.s/m for a similar three-story frame using GA alone, indicating that the IWD algorithm achieves a 5.3% reduction in required damping. Aydin et al. [17] reported optimal damping coefficients in the range of 200–350 kN.s/m for rehabilitated planar frames of comparable stiffness. The second-story-dominant distribution pattern observed herein is also consistent with the findings of Takewaki [3] and Pollini [15], who demonstrated that optimal damper placement in low-rise moment frames correlates with the interstory drift profile of the fundamental mode.

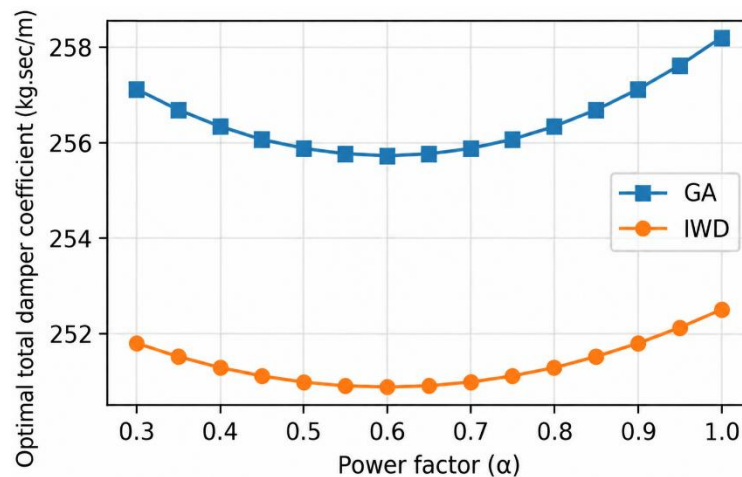


Fig. 14. Sensitivity of optimal total damper coefficient to VD power factor α (GA vs IWD).

7. Conclusions

This study investigated the optimal distribution of fluid viscous dampers (FVDs) in a three-story steel moment-resisting frame using the endurance time (ET) method and compared the performance of the genetic algorithm (GA) and intelligent water drops (IWD) optimization algorithms under identical modeling and optimization conditions. Based on the results obtained, the following conclusions can be drawn:

1. Both GA and IWD consistently identified the second story as the most effective location for concentrating the supplemental damping. Approximately 64–66% of the total optimized damping coefficient was assigned to the second story, indicating that the optimal damper distribution is strongly related to the deformation characteristics of the structure.
2. The IWD algorithm provided better optimization performance than GA for the investigated structural system. The IWD-based solution resulted in approximately 6–9% lower mean peak interstory drift while requiring a slightly lower total damping demand. The IWD algorithm also exhibited smoother convergence during the optimization process.
3. The optimized total damping coefficient was 250.9 kN.s/m. The corresponding story-level distributions showed that the dominant contribution of the supplemental damping was maintained at the second story for both BSE-1 and BSE-2 hazard levels.
4. Comparison with conventional nonlinear time-history analysis showed that the ET-based predictions remained within approximately 12% of the corresponding NLTHA results. This agreement supports the applicability of the ET method as an efficient screening approach for iterative performance-based optimization of supplemental damping systems.
5. The optimized FVD system dissipated approximately 35–45% of the seismic input energy, resulting in a reduction in the nonlinear demand imposed on the primary structural members. The peak plastic hinge rotation was also reduced from 0.021 rad in the bare frame to 0.012 rad after damper installation under the BSE-2 level.
6. The sensitivity analysis demonstrated that increasing the velocity exponent of the viscous dampers can reduce peak interstory drift while increasing peak floor acceleration. This finding highlights the importance of considering both displacement- and acceleration-based performance measures when selecting damper properties.

Overall, the results demonstrate that the combination of the ET method with IWD optimization can provide an effective computational framework for the performance-based distribution of FVDs in low-rise steel frames. The findings are limited to the investigated three-story benchmark structure; therefore, further studies should examine the applicability of the proposed framework to taller, irregular, and different structural systems and should consider the simultaneous optimization of damper distribution and nonlinear velocity parameters.

Statements & declarations

Author contributions

Mehdi Shalchi Tousi: Conceptualization, Methodology, Resources, Writing - Original Draft.

Zakia Emrany: Investigation, Formal analysis, Resources, Writing - Original Draft.

Payam Tarighi: Formal analysis, Project administration, Writing - Review & Editing.

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Data availability

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

Declarations

The authors have no conflicts of interest to declare that are relevant to the content of this article.

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Axial Performance of Reinforced Concrete Columns with Lap Splices and Flash-Butt Welded Splices under Concentric Compression

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ABSTRACT

This study presents an extensive experimental investigation into the axial behavior of reinforced-concrete columns containing different splice types, transverse reinforcement configurations, and cross-sectional geometries. A total of six groups of circular and square columns were tested under concentric compression to evaluate the structural performance of lap splices and flash-butt (forging) splices in combination with various confinement systems, including individual ties, circular hoops, and spirals with different pitches. Mechanical and electrical strain gauges, displacement transducers, and load cells were employed to monitor stresses in longitudinal and transverse reinforcement as well as concrete strains at multiple heights, enabling a detailed assessment of stiffness degradation, crack initiation, ductility, and failure mechanisms. The results demonstrate that the measured peak axial loads ranged from 371.52 kN for the plain-concrete reference specimen to 958.48 kN for specimen FB. The pre-peak secant stiffness at 40% of peak load, $K_{0.4}$, ranged from 23.73 to 129.25 kN/mm, whereas the normalized energy-absorption ratio ranged from 0.45 to 1.10. Specimen FB, which contained a bottom flash-butt-forged splice and individual hoops, achieved the highest peak load, stiffness, and energy-absorption ratio. However, no consistent overall superiority of either lap splices or forged splices was observed; the response depended on the interaction among splice type, splice location, confinement, concrete strength, and specimen-specific construction quality. Specimens without transverse reinforcement in the splice region generally exhibited earlier cracking, wider cracks, and less stable deformation. The maximum crack width, 3.0 mm, was recorded for specimen LSM. Reducing the spiral pitch from 95 mm to 50 mm generally improved cracking resistance and deformation stability, although the magnitude of improvement varied among specimens. The results demonstrate that adequate transverse confinement and reliable construction quality are essential for stable axial performance of columns containing either lap splices or flash-butt-forged splices.

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1. Introduction

Reinforced concrete (RC) is the most widely used structural material in modern construction; concrete's limited tensile capacity is compensated by steel reinforcement, which frequently must be spliced because of bar-length limits and construction sequencing. Traditional lap splices overlapping bars for a specified length with transverse ties are simple, well codified and widely used, but they require additional length and can complicate detailing in congested regions [1–3]. Flash-butt forging (butt welding) produces a

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continuous metallic connection at bar ends and can reduce required splice length and ease congested detailing; however, metallurgical and geometric changes in the weld zone and the local stress-transfer mechanism raise questions about ductility, fatigue and post-peak behavior, particularly for members expected to form plastic hinges under seismic loading [4-11].

Parametric laboratory studies and code commentaries indicate that lap-splice capacity and post-peak ductility are governed primarily by lap length, bar diameter, concrete strength, concrete cover, and the level of transverse confinement, and that increasing lap length or improving confinement (tighter ties or spiral) consistently improves both strength and deformation capacity [1, 2, 12]. Controlled laboratory implementations and comparative evaluations show that properly executed forged splices can achieve static strength comparable to well-detailed laps in monotonic tests, but forged splices are more sensitive to welding procedure, heat-affected-zone (HAZ) effects, and fatigue performance; therefore, qualification, post-weld treatment, and non-destructive inspection are essential before their use in ductility-critical regions [3-7, 11].

A substantial body of experimental work demonstrates that continuous spiral confinement or closely spaced ties substantially increase compressive strain capacity and ductility of columns, delaying core crushing and stabilizing post-peak softening; enhanced confinement also reduces the performance gap between lap and welded splices, so that welded splices enclosed by strong confinement can approach the ductility of well-detailed lap splices provided the splice itself is metallurgically sound [2, 12-14]. Practical welding and inspection studies further emphasize the need for weld qualification: ultrasonic and other non-destructive testing methods have been proposed and applied to gas-pressure and flash-butt welded joints to detect defects in the HAZ and ensure reliable field acceptance [1, 7, 8, 11, 13, 15].

Country-level experience supports these laboratory findings. Japanese programs testing gas-pressure/flash-butt welded rebars reported that properly executed welds satisfied national tensile and bending requirements and that failures in many tests occurred away from the forged section, supporting the structural adequacy of qualified forging procedures; nevertheless, these programs also stress the importance of post-weld inspection and workmanship control, and recommend immediate post-weld finishing and stricter weld control as practiced after the 1995 Hanshin earthquake [7-9, 12]. Complementary experimental studies show that tighter spiral pitch or closer stirrup spacing increases ultimate load and ductility and can change failure modes (core crushing, spiral fracture, or longitudinal buckling), while full-scale cyclic and axial-cyclic tests confirm improved post-buckling and cyclic performance with enhanced confinement and with hybrid reinforcement schemes [10, 12, 16, 17].

Practical economy and detailing studies indicate welded splices can be economical for smaller bars when welding procedures and lengths are adjusted with bar diameter, but they also highlight the need for procedural qualification and inspection to manage HAZ and fatigue risks; review studies of weld fatigue and comparative welding methods reinforce the call for standardized qualification and inspection protocols [6, 7, 11, 18]. Research on strengthening and retrofit methods (FRP, external spirals, grooving/EBR techniques and hybrid steel-FRP reinforcement) further shows that confinement and external reinforcement substantially affect column capacity and ductility, underscoring that splice performance cannot be considered in isolation from confinement or retrofit measures [17, 19-21].

Recent experimental and analytical studies have confirmed that the structural behavior of reinforced-concrete members containing lap splices is governed by the interaction among lap-splice length, reinforcement diameter, concrete compressive strength, concrete cover, bond conditions, transverse reinforcement, and loading history [22-24]. Adequate transverse confinement is particularly important in lap-splice regions because it can improve bond transfer, delay splitting cracks, limit cover spalling, enhance post-peak response, and increase the deformation capacity of reinforced-concrete members [22-24]. Recent experimental research has also emphasized that lap-splice performance in reinforced-concrete columns is highly dependent on the confinement level and the detailing of the splice region, especially when the column is subjected to large inelastic deformation demands [23, 24].

The performance of flash-butt-forged or welded reinforcing bars is also dependent on factors beyond the nominal tensile capacity of the reinforcement. Recent investigations have shown that welding parameters, heat-affected-zone characteristics, local geometric irregularities, weld defects, and fatigue resistance can affect the mechanical reliability of welded reinforcing bars [3, 4, 11]. In particular, recent research on high-strength reinforcing bars joined by flash-butt welding has highlighted the importance of welding quality control, post-weld inspection, and evaluation of fatigue behavior before application in structural members subjected to cyclic or inelastic loading [3, 4]. Therefore, a welded or forged splice that satisfies a static tensile-strength requirement may still require careful assessment of its local stress transfer, ductility, and long-term performance.

Recent studies on reinforced-concrete columns further confirm that the transverse-reinforcement type, spacing, and configuration strongly influence the confinement effectiveness, concrete-core behavior, reinforcement stability, axial deformation capacity, and post-peak response of the members [14, 17]. Closer spiral pitch or denser transverse reinforcement generally improves confinement of the concrete core, delays crushing and spalling, and promotes a more gradual reduction in load after the peak response. Conversely, insufficient confinement can lead to premature deterioration of the concrete core, increased transverse reinforcement demand, reinforcement instability, and brittle failure [14, 17].

Although the effects of lap splices, welded reinforcement, and transverse confinement have been studied independently, relatively limited experimental information is available on the direct comparison of lap-spliced and flash-butt-forged reinforcing bars in short reinforced-concrete columns subjected to monotonic concentric compression. In particular, the combined effects of splice type, splice location, transverse-reinforcement configuration, confinement spacing, and cross-sectional shape on axial capacity, reinforcement stress, cracking response, post-peak behavior, and energy-based ductility require further investigation. Accordingly, the present study evaluates the axial performance of 22 short reinforced-concrete columns containing lap splices or

flash-butt-forged splices under monotonic concentric compression.

The experimental program consisted of 22 short RC columns arranged in six test groups, designed to isolate the effects of splice type (lap vs. flash-butt-forged), transverse confinement (stirrup spacing, spiral vs. ties), and section/reinforcement detailing. Specimens were instrumented to record longitudinal and transverse rebar stresses, concrete strain, crack widths, axial and lateral displacements, and peak load; failure modes and failure lengths were documented by visual inspection and photographic records. Test variables included bar diameter and arrangement, concrete strength, confinement configuration, and splice location; monotonic axial compression tests were performed to obtain peak and post-peak behavior.

The study aims to: (1) quantify the difference between theoretical capacity and measured peak load for lap and forged splices under axial compression; (2) compare ductility and post-peak softening as a function of splice quality and transverse confinement; and (3) evaluate the effectiveness of confinement detailing (spiral vs. ties, spacing) in restoring or improving ductility of welded splices.

2. Experimental program

2.1. Objectives and scope

The experimental program was designed to compare the axial compressive response, failure modes, and ductility of short RC columns containing either lap splices or flash-butt forged splices. The study varied cross-section geometry, transverse reinforcement type and spacing, and splice location to evaluate the influence of confinement and detailing on splice performance.

2.2. Specimen matrix and variables

A total of 22 short RC columns were cast and tested, arranged into six groups. The principal variables were:

- Splice type: lap splice (overlap) vs. flash-butt forging (butt weld/forging).
- Cross-section shape: square vs. circular.
- Transverse reinforcement: closed ties (stirrups) vs. continuous spiral confinement; spacing and diameter varied.
- Splice location: mid-height (central) vs. near the column end (top or bottom).

All reinforced-concrete columns were longitudinally reinforced with 16-mm deformed bars and transversely reinforced with 8-mm deformed ties. Tensile tests were conducted on representative samples of each bar diameter in accordance with ASTM A615 [25]. The measured mean yield strength, ultimate tensile strength, and modulus of elasticity are summarized in **Error! Reference source not found.** These measured properties were used to assess whether the stresses recorded by the strain-gauge instrumentation reached the yield level during the tests.

Table 1. Mechanical properties of the reinforcing steel used in the test specimens

Reinforcement type	Nominal diameter (mm)	Steel grade	Yield strength, f_y (MPa)	Ultimate tensile strength, f_u (MPa)	Elastic modulus, E_s (GPa)
Longitudinal reinforcement	16	Grade 60	420	600	200
Transverse reinforcement: ties/hoops/spirals	8	Grade 40	300	500	200

Longitudinal and transverse reinforcement spacing and detailing follow the provisions of ACI 318-19 [1], and the concrete cover for all specimens was maintained 20 mm. The nominal concrete cover was controlled during reinforcement-cage assembly by means of 20 mm spacers positioned around the reinforcement cage before casting. The same nominal cover-control procedure was adopted for all specimens to minimize variability in the effective core dimensions and reinforcement location. This constitutes a limitation of the experimental program and is acknowledged when interpreting specimen-to-specimen differences. Each group used the same concrete mix and curing regime for all specimens in that group; specimens differed only in splice type/location and transverse reinforcement spacing. Detailed descriptions for each group are provided below.

Group 1 comprised four circular reinforced-concrete columns, each reinforced with four longitudinal bars and separate circular ties (individual hoops) spaced at approximately 95 mm. Two specimens contained 300 mm lap splices, one located at mid-height and the other at the column base, while the remaining two specimens used flash-butt (forged) splices, likewise placed one at mid-height and one at the base. All specimens in this group shared identical concrete mix and environmental curing conditions; the geometric and reinforcement details are shown in Fig. 1.

Each column name encodes the splice type and splice location. FB: forging at Bottom (flash butt splice located at the column base), FM: forging at mid height (flash butt splice located at mid height). SB: splice (Lap) at bottom (300 mm lap splice located at the column base). SM: splice (Lap) at mid height (300 mm lap splice located at mid height).

Group 2 consists of four circular reinforced-concrete columns geometrically and materially identical to those in group 1 but with the transverse reinforcement intentionally omitted at the splice region (Fig. 2).

The specimens in this group are designated LFB (forging at bottom), LFM (forging at mid height), LSB (lap splice at bottom), and LSM (lap splice at mid height), where each code indicates the splice type (forging or lap) and its location along the column height.

Group 3 consists of four circular reinforced-concrete columns identical in geometry and materials to group 2, but with continuous spiral confinement used for the transverse reinforcement at a pitch of 95 mm (Fig. 3). The use of closely spaced spiral confinement in this group is intended to investigate the influence of enhanced transverse confinement on splice performance, ductility, and failure mode under the applied loading protocol.

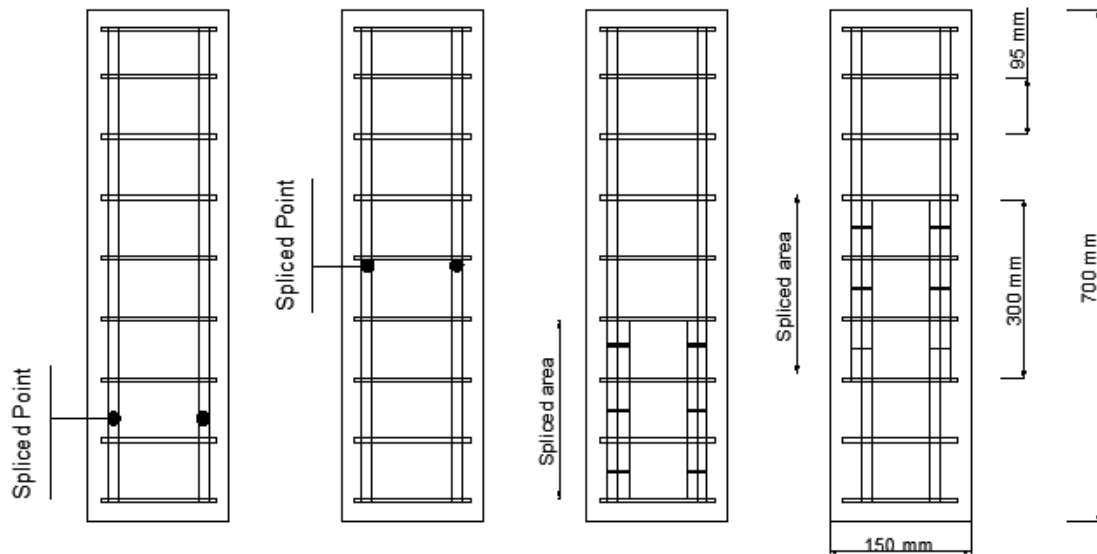


Fig. 1. Location, splice type, and tie details for group 1.

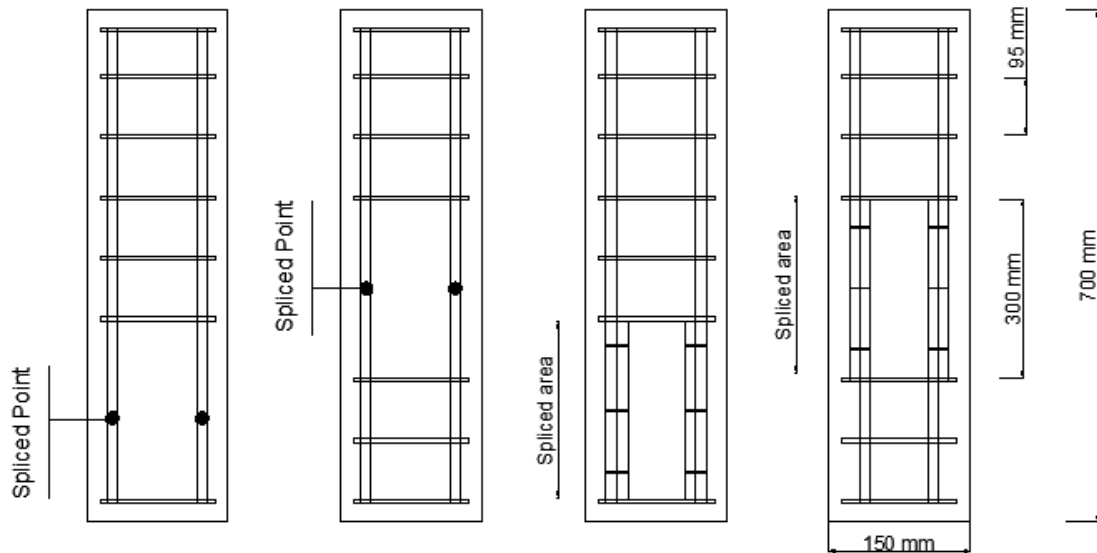


Fig. 2. Location, splice type, and tie details for group 2.

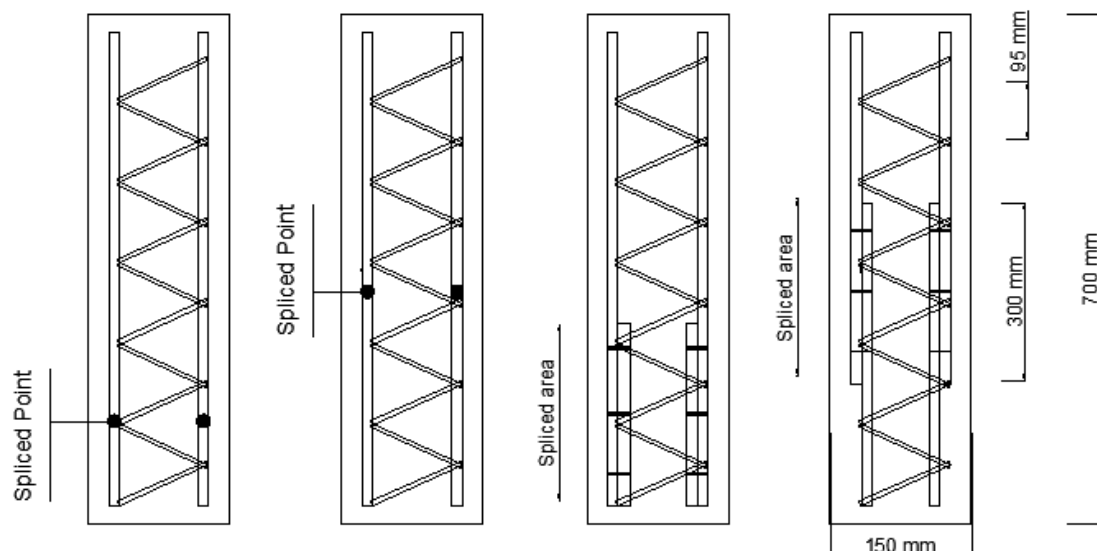


Fig. 3. Location, splice type, and the transverse spiral reinforcement details for group 3.

The specimens in group 3 are designated SFB (forging at bottom), SFM (forging at mid-height), SSB (lap splice at bottom), and SSM (lap splice at mid-height).

Group 4 comprises four circular columns similar to those in group 3 but with the spiral pitch reduced to 50 mm to provide higher transverse confinement; the specimens are designated HSFB (forging at bottom), HSBM (forging at mid-height), HSSB (lap splice at bottom), and HSSM (lap splice at mid-height). Group 5 contains four square columns whose tie arrangement and splice locations mirror those of group 1; these specimens are labeled RFB (forging at bottom), RFM (forging at mid-height), RSB (lap splice at bottom), and RSM (lap splice at mid-height). Group 6 includes two circular specimens used as control columns: an unreinforced concrete column denoted as PC, and a reinforced circular column with continuous (unspliced) longitudinal bars and individual hoops denoted as L.

2.3. Materials and fabrication

Normal-weight concrete with controlled mix proportions was used for all specimens. A crushed angular coarse aggregate with a nominal particle-size range of 6–12 mm and fine aggregate with a maximum particle size of 4 mm was also used in the concrete mixtures. The use of a maximum coarse-aggregate size of 12 mm was considered suitable for the relatively small cross-sectional dimensions of the test columns, the nominal concrete cover, and the locally congested reinforcement and splice regions, thereby facilitating concrete placement and consolidation around the reinforcement cage. The physical properties are summarized in Table 2. Coarse-aggregate and fine-aggregate properties were determined in accordance with ASTM C127 [26] and ASTM C128 [27], respectively.

Table 2. Physical properties of the fine and coarse aggregates used in the concrete mixture.

Aggregate type	Particle-size range (mm)	Specific gravity	Water absorption (%)	Bulk density (kg/m ³)
Fine aggregate	≤ 4	2.5	0.5	2.6
Coarse aggregate	6–12	2.7	1.5	2.6

Compressive strength f'_c was measured on companion cylinders at the time of testing; average values for each batch were recorded. Deformed steel bars of specified diameters were used. Lap lengths were chosen according to code provisions for lap splices; forging/welded splices were produced by flash-butt forging equipment with controlled heating and forging parameters. Either closed rectangular ties or continuous spiral reinforcement was provided. Spiral confinement was used in selected circular specimens to study confinement effects. Lap splices were prepared by overlapping bars for the design lap length and tying with binding wire. Flash-butt forging splices were produced by aligning bar ends, applying electric flash heating and axial pressure to forge the ends into a continuous section. Welding/forging parameters and visual inspection records were kept (Fig. 4).



Fig. 4. (a) Longitudinal clamped rebars prior to heating; (b) heating procedure; (c) Rebars spliced by the forging method; and (d) reinforcement cages of the specimens.

Circular specimens were cast using PVC tubes with an outer diameter of 160 mm, while square specimens were cast in a custom-designed, removable steel formwork (Fig. 5). 20 mm spacers were used to ensure the specified concrete cover and uniform cover thickness. To facilitate casting and vibration, all forms were secured in a laboratory holding device that had been fixed to the floor with gypsum. A single concrete mix design was used for all groups; however, companion specimens for compressive strength tests were taken at each casting stage. Although a common nominal concrete mix design and curing regime were adopted for the experimental program, separate casting stages were required for specimen fabrication. Therefore, companion concrete specimens were prepared and tested for each casting stage to determine the compressive strength corresponding to the relevant column specimens at the time of testing. The measured concrete compressive strength, associated with each specimen is reported in Table 3 and was used in calculating the theoretical axial capacity of that specimen. Consequently, the theoretical capacities reported in Table 4 reflect the measured concrete strength rather than a single assumed compressive-strength value for all columns. For each casting campaign, at least three 150 × 150 × 150 mm cubes and three 150×300 mm cylinders were cast and tested for capacity calculations. One day after casting, specimens were demolded and cured under identical moist conditions until the test age.



Fig. 5. Filled column formworks after placement and vibration.

Table 3. Numerical summary of all tested specimens.

Specimen	f'_c (MPa)	Service load $P_s = 0.4P_c$ (kN)	Cracking Load (kN)	Transverse reinforcement	Spacing (mm)
SM	29.2	206.4	562.0	Individual hoops	95
SB	33.8	238.9	630.0	Individual hoops	95
FM	30.6	216.3	297.6	Individual hoops	95
FB	33.8	238.9	724.0	Individual hoops	95
LSM	34.5	243.8	124.0	Hoops omitted at splice	95
LSB	23.1	163.3	230.0	Hoops omitted at splice	95
LFM	34.5	243.8	120.0	Hoops omitted at splice	95
LFB	27.5	194.4	297.0	Hoops omitted at splice	95
SSM	31.8	224.7	102.0	Spiral	95
SSB	22.5	159.0	207.0	Spiral	95
SFM	22.5	159.0	410.0	Spiral	95
SFB	31.8	224.7	310.0	Spiral	95
HSSM	31.5	222.6	700.0	Spiral	50
HSSB	31.5	222.6	282.4	Spiral	50
HSFM	32	226.2	358.0	Spiral	50
HSFB	26.4	186.6	351.7	Spiral	50
RSM	28	197.9	262.0	Closed ties	95
RSB	37.6	265.8	390.0	Closed ties	95
RFM	34.5	243.8	490.0	Closed ties	95
RFB	18.2	128.6	351.7	Closed ties	95
L	22.5	159.0	162.0	Individual hoops	95
PC	41	289.9	—	None	—

2.4. Instrumentation and test setup

Strain gauges were attached to longitudinal and transverse bars and to concrete surfaces to monitor axial strains. Concrete axial strain was monitored using electrical resistance strain gauges attached to the external concrete surface of the specimens. The gauges were oriented parallel to the column longitudinal axis and installed at mid-height and near ends. The concrete strain values reported in Table 4 correspond to the maximum measured surface strain. Because the gauges were attached to the external surface, the recorded values represent local axial strains in the cover concrete at the instrumented locations. They were used to monitor local deformation and cracking but do not directly represent the axial strain of the confined concrete core inside the transverse reinforcement.

The electrical strain recorded by each gauge was converted to steel stress using the adopted bilinear elastic-perfectly plastic stress-strain relationship. In the elastic range, the reinforcement stress was calculated as Eq. 1:

$$\sigma_s = \begin{cases} E_s \varepsilon_s & \text{if } |E_s \varepsilon_s| < f_y \\ f_y & \text{if } |E_s \varepsilon_s| \geq f_y \end{cases} \quad (1)$$

where σ_s is the local reinforcement stress, E_s is the elastic modulus of the reinforcing steel, and ε_s is the strain recorded by the strain gauge.

The modulus of elasticity was taken as $E_s = 200$ GPa. The yield strength of the reinforcement was $f_y = 420$ MPa.

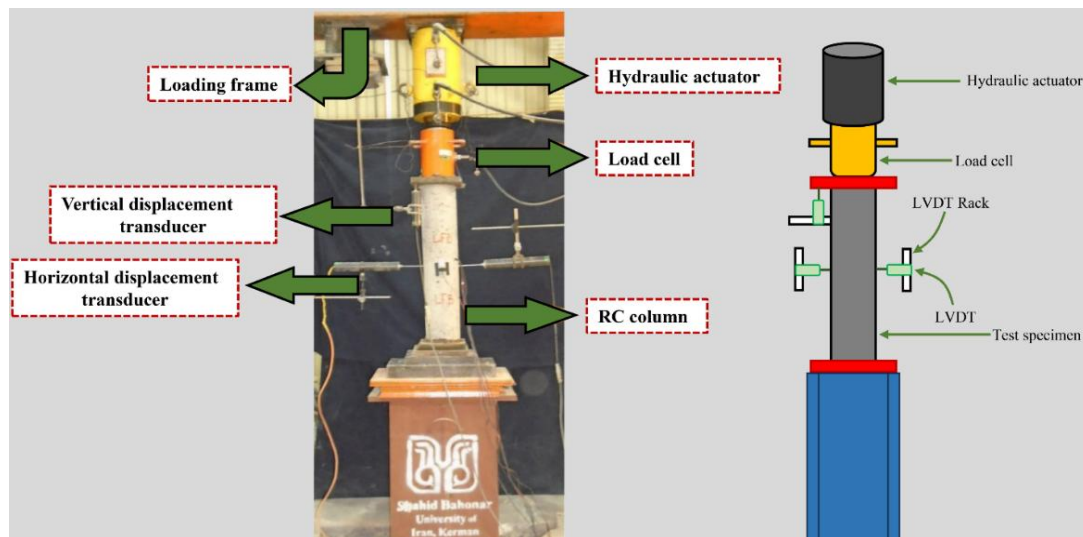
The stresses shown in Table 4 correspond to the local gauge-derived stress at the experimentally measured peak axial load (P_{exp}). When more than one gauge was installed on the same reinforcement category, the reported value represents the maximum gauge at the critical location. Gauge readings obtained after gauge debonding, wire damage, or data loss were excluded from the stress calculation.

Table 4. Comparative summary of theoretical capacity, stresses, strains, and deformations for tested columns.

Specimen	Theoretical capacity P_{th} (kN)	Experimental peak load P_{exp} (kN)	P_{th}/P_{exp}	Longitudinal rebar stress (MPa)	Transverse rebar stress (MPa)	Concrete strain	Failure Length (mm)	Crack Width (mm)	Axial Displacement (mm)	Lateral displacement (mm)
SB	713.0	817.3	0.87	434.0	102.0	0.0014	310	0.16	10.87	2.83
FB	713.0	977.4	0.73	312.0	226.2	0.0015	290	0.06	8.25	6.62
SM	660.2	825.0	0.80	297.8	124.0	0.0009	335	0.18	9.83	2.42
FM	676.9	739.2	0.92	308.6	353.8	0.0007	300	0.20	11.08	3.03
LSB	590.3	630.9	0.94	243.8	453.0	0.0004	250	1.80	10.47	3.07
LFB	640.2	547.1	1.17	283.2	283.2	0.0008	280	2.00	10.46	0.98
LSM	721.6	741.0	0.97	340.0	260.0	0.0005	210	3.00	11.48	—
LFM	721.6	615.7	1.17	281.6	398.2	0.0009	165	0.22	12.34	3.24
SSB	600.0	470.5	1.28	242.2	62.2	0.0013	370	0.20	6.87	2.02
SFB	713.6	610.0	1.17	356.2	387.6	0.0023	280	0.20	9.66	1.98
SSM	713.6	636.2	1.12	161.4	332.2	0.0012	310	1.80	12.43	4.70
SFM	600.0	541.0	1.11	272.4	69.8	0.0005	390	0.04	10.83	4.58
HSSB	598.5	620.0	0.97	210.4	158.2	0.0005	270	0.20	10.05	1.90
HSFB	647.3	687.0	0.94	286.4	290.8	0.0006	220	1.40	12.10	2.18
HSSM	709.4	795.3	0.89	242.8	174.4	0.0017	200	1.80	10.85	3.32
HSFM	551.0	574.2	0.96	250.6	188.6	0.0007	230	0.20	11.13	2.68
RSB	880.0	526.6	1.67	115.0	85.6	0.0008	460	0.24	10.30	2.33
RFB	592.5	764.9	0.77	312.0	296.0	0.0014	300	0.04	12.10	2.18
RSM	784.3	804.5	0.97	355.8	415.6	0.0012	350	0.40	9.34	2.60
RFM	834.4	732.6	1.14	296.0	425.4	0.0009	290	0.14	11.25	3.85
PC	492.6	396.6	1.24	—	—	0.0008	460	—	—	—
L	579.7	614.0	0.94	233.8	199.0	0.0009	265	0.14	11.01	2.82

Note: missing or unrecorded values are shown as —.

Displacement transducers measured axial shortening and lateral deformations. Crack monitoring and crack-width gauges recorded crack initiation and propagation. Data acquisition logged all channels continuously during monotonic concentric axial loading to failure. A hydraulic actuator applied concentric axial load under monotonic protocol until failure (Fig. 6).

**Fig. 6. Filled column formworks after placement and vibration.**

To ensure uniform load distribution and improve test reliability, all circular specimens were double-capped with sulfur caps, thereby ensuring that the applied axial load was distributed evenly over the entire specimen cross-section. Prior to each test, the specimen was manually centered on the lower loading platen so that its centroidal axis approximately coincided with the actuator loading axis. For circular specimens, sulfur capping was applied at both ends to provide level and approximately parallel bearing surfaces, thereby improving load distribution and reducing local stress concentrations at the specimen ends. A small seating load was applied before the main test to establish full contact between the specimen and the loading platens. The specimen position was visually checked and adjusted, where necessary, before the monotonic axial load was applied. Although the alignment procedure was carefully controlled, the initial loading eccentricity was not instrumentally measured for each specimen.

3. Results and discussions

3.1. General observations and failure modes

Failure location: almost all specimens failed in the upper region of the column (except one specimen), even when cracking initiated at the bottom (Fig. 7). This indicates that local conditions and stress redistribution influenced ultimate failure location. The observed concentration of failure in the upper region of many specimens may be related not only to local concrete quality, splice location, and end conditions, but also to small unavoidable deviations from ideal concentric loading. A residual loading eccentricity can introduce a secondary bending effect under nominally axial compression, resulting in nonuniform concrete stress, unequal stress development in the longitudinal reinforcement, and localized crack initiation or crushing. Because initial eccentricity was not measured instrumentally for each specimen, its individual contribution cannot be quantified. Nevertheless, the consistent loading setup, sulfur-capping procedure, and specimen-centering method applied to all specimens reduce the likelihood that alignment differences alone explain the overall comparative trends among the tested groups.



Fig. 7. Filled column formworks after placement and vibration.

Crack patterns: axial compression produced vertical splitting cracks; confinement detailing affected crack width and localization. Spiral-confined specimens showed more distributed cracking and delayed localization.

Concrete strain at failure: circular columns with spiral confinement reached concrete strains exceeding 0.003, surpassing the commonly referenced ACI nominal crushing strain for unconfined concrete.

Longitudinal steel behavior: in nearly all specimens, the longitudinal reinforcement did not reach yield at peak load, indicating that concrete crushing or confinement-controlled failure governed ultimate capacity.

The predominance of top-region failures suggests that local stress concentrations, slight eccentricities, or splice location relative to end conditions influenced ultimate behavior. The observation that longitudinal bars generally did not yield implies that concrete crushing or confinement-controlled failure governed the ultimate capacity in most specimens. This is consistent with confined concrete theory: when confinement is insufficient, concrete crushing can occur before steel yields.

3.2. Representative experimental summary

The representative numerical summary of all tested specimens shown in Table 3 provides a clear comparative view of how confinement type, splice detailing, and concrete quality influence the axial performance of reinforced-concrete columns. The measured concrete compressive strength was not uniform among all specimens; the recorded values ranged from 18.2 MPa to 41.0 MPa. Therefore, direct comparison of raw cracking load, peak load, and deformation capacity across all specimens would not isolate the effects of splice type or transverse-reinforcement configuration. To account for this variation, the theoretical axial capacity of each specimen was calculated using its corresponding measured concrete compressive strength, and the experimental response was interpreted together with the experimental-to-theoretical capacity ratio, P_{exp}/P_{th} . Comparisons between splice types and confinement details were also prioritized for specimens with comparable geometry, reinforcement arrangement, splice location, and concrete strength. Nevertheless, residual variation in concrete quality and strength remains a source of scatter in the experimental results.

As shown in Table 3, specimens in group 1, which were confined with individual hoops at 95 mm spacing, consistently exceeded their theoretical load capacities, with increases ranging from 9% to 32%, demonstrating the effectiveness of proper transverse reinforcement. In contrast, specimens in group 2, where hoops were intentionally omitted over the splice region, exhibited noticeable reductions in ultimate strength up to 15% below theoretical predictions and significantly larger crack widths, reaching 3 mm in LSM. This confirms that the absence of confinement in splice zones directly compromises structural integrity and accelerates failure.

Columns in group 3, reinforced with spirals at a relatively large pitch of 95 mm, also failed below their theoretical capacities (by 10–22%), largely due to insufficient confinement. Their failure lengths, often exceeding 30–39 cm, indicate extensive damage propagation. In contrast, group 4, with a reduced spiral pitch of 50 mm, demonstrated markedly improved cracking resistance such as the 700 kN cracking load in HSSM, highlighting the substantial benefits of dense spiral confinement. Square columns in group 5 showed competitive performance, with cracking loads up to 490 kN, reflecting the effectiveness of closed ties in rectangular sections. Finally, the reference specimens in group 6 illustrate baseline behavior: the plain concrete column (PC) exhibited high compressive strength but no ductility, while the continuously reinforced column (L) cracked at 1620 kN, emphasizing the role of splice detailing and confinement in enhancing column performance.

The table below consolidates the principal experimental outcomes for all tested columns across six groups. Each row corresponds to a single specimen and reports the theoretical axial capacity, measured peak load, central concrete strain (digital), and the experimental/theoretical capacity ratio. This condensed dataset is intended to support direct comparison of transverse reinforcement detailing, splice/connection type, and concrete quality on ultimate capacity and ductility. The paragraphs that follow interpret these results and draw engineering conclusions and recommendations.

The consolidated experimental dataset shows a clear and consistent pattern: most specimens failed by concrete crushing at the compression face, while longitudinal reinforcement generally remained below yield at peak load. Measured peak loads ranged widely (≈ 397 – 977 kN), and the experimental-to-theoretical capacity ratio varied from about 60% to 137%, indicating that detailing and material integrity produced both substantial capacity shortfalls and notable reserves relative to theoretical predictions.

Transverse confinement emerged as the primary control on both strength and ductility. Specimens with close spiral pitch or dense ties recorded higher central strains, smaller failure lengths, and more gradual post-peak softening, demonstrating that effective confinement raises the compressive strain capacity of the core and delays catastrophic core crushing. In these well-confined columns, the axial shortening at peak was larger, and the residual capacity after peak load was more pronounced.

Concrete quality and workmanship had an equally decisive effect. Columns exhibiting honeycombing or poor compaction (notably RSB and several group 3 specimens) showed the largest reductions in peak load – up to roughly 40% below theoretical – and developed broader, more diffuse failure zones. Surface repairs did not reliably restore core integrity, so poor placement and consolidation produced permanent losses in both capacity and deformation capacity.

Splice type alone did not determine ultimate performance; rather, splice behavior depended on the interaction of splice form, local detailing, confinement, and fabrication quality. At service levels, lap and mechanical/forged splices behaved similarly when confinement was comparable, but at ultimate loads lap splices combined with adequate confinement tended to produce more progressive, ductile failures. Mechanical splices achieved high peak loads in some cases but were more sensitive to fabrication defects and therefore require strict quality assurance to ensure reliable performance.

Observed stress patterns underline the demand placed on transverse reinforcement: several specimens showed transverse bar stresses approaching or exceeding yield while longitudinal bars remained elastic, indicating significant lateral expansion of the core under axial compression. Where ties or spirals were insufficient, tie rupture or opening precipitated rapid core disintegration; this highlights the need to design transverse reinforcement not only for confinement but also to resist the lateral pressures that develop prior to longitudinal yielding.

From a practical standpoint, the tests support three immediate recommendations: provide robust confinement around splice zones (close spiral pitch or closely spaced ties); enforce rigorous quality control for mechanical splices (factory certification, sample destructive tests and nondestructive inspection) and combine splices with confinement detailing; and ensure proper concrete placement and compaction, since workmanship defects cannot be reliably corrected by surface patching.

The study's limitations should be acknowledged: tests were monotonic axial compression on short columns, so behavior under combined axial-bending loading or cyclic seismic actions may differ. A statistical grouping of specimens (means and standard deviations by splice and confinement type) and targeted tests that vary splice location, splice length, and controlled fabrication defects would strengthen the conclusions and refine design guidance.

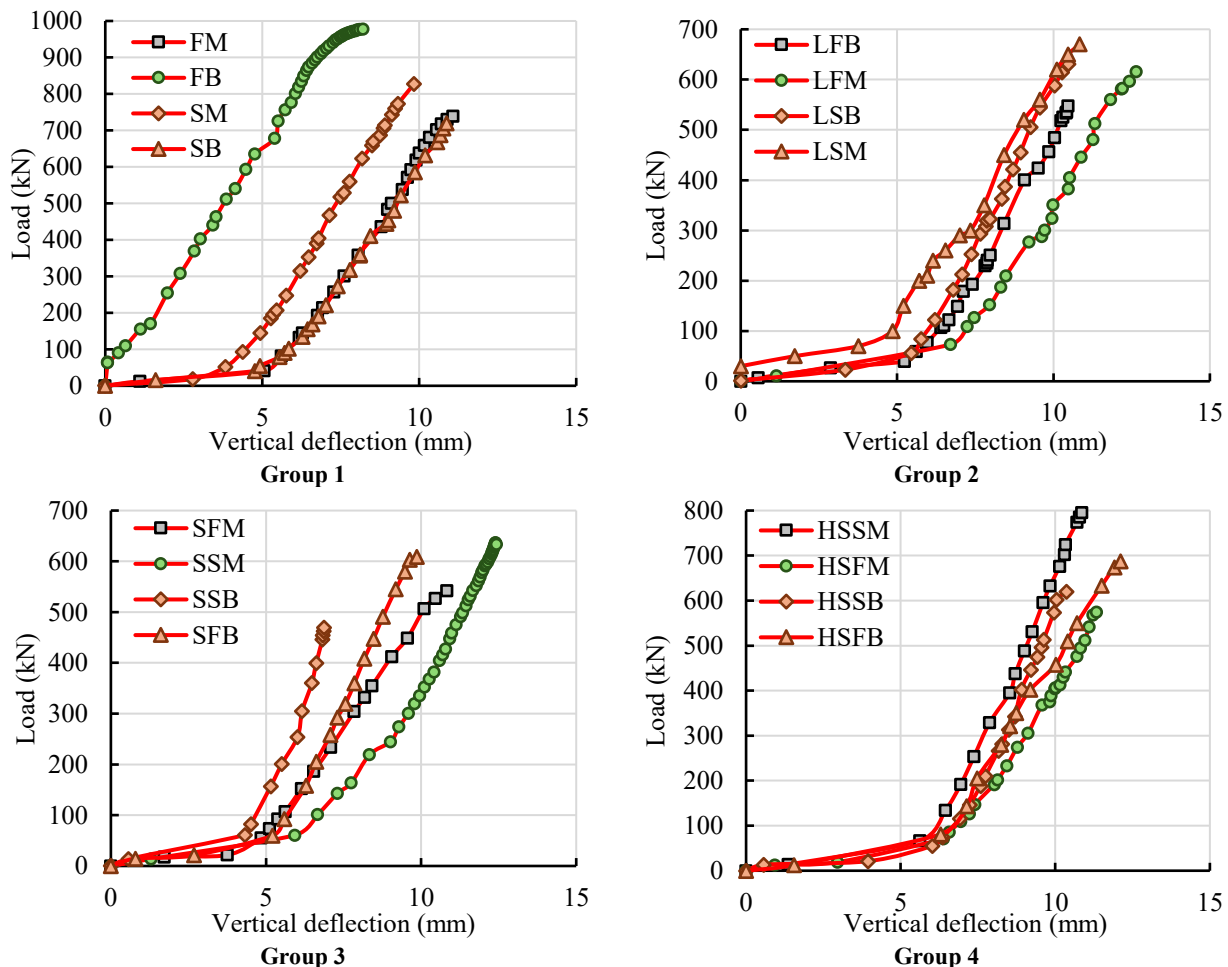
In summary, the experimental evidence is unequivocal: transverse confinement and concrete workmanship are the dominant determinants of column strength and ductility. Splice selection must be considered within that broader detailing and quality framework—adequate confinement and strict quality control are the decisive measures to ensure safe, ductile column performance.

Overall, the numerical summary reveals a consistent pattern: adequate confinement, especially with reduced spiral pitch, significantly increases cracking load, reduces damage length, and improves ultimate strength, whereas insufficient confinement or poor concrete quality can reduce capacity by more than 20% and lead to wider cracks and brittle failure. This comparative dataset provides a strong quantitative foundation for evaluating the structural reliability of different splice and confinement configurations.

3.3. Vertical load-deflection response

The vertical load-deflection diagrams for all column groups are presented in Fig. 8, illustrating the deformation behavior of the specimens under axial loading.

The vertical load-deflection curves derived from the experimental data provide a comprehensive view of the axial stiffness, nonlinear transition, and failure behavior of all tested column groups. To quantify the axial stiffness of the tested specimens, secant stiffness values were determined from the ascending branch of each experimental axial load-axial displacement curve.



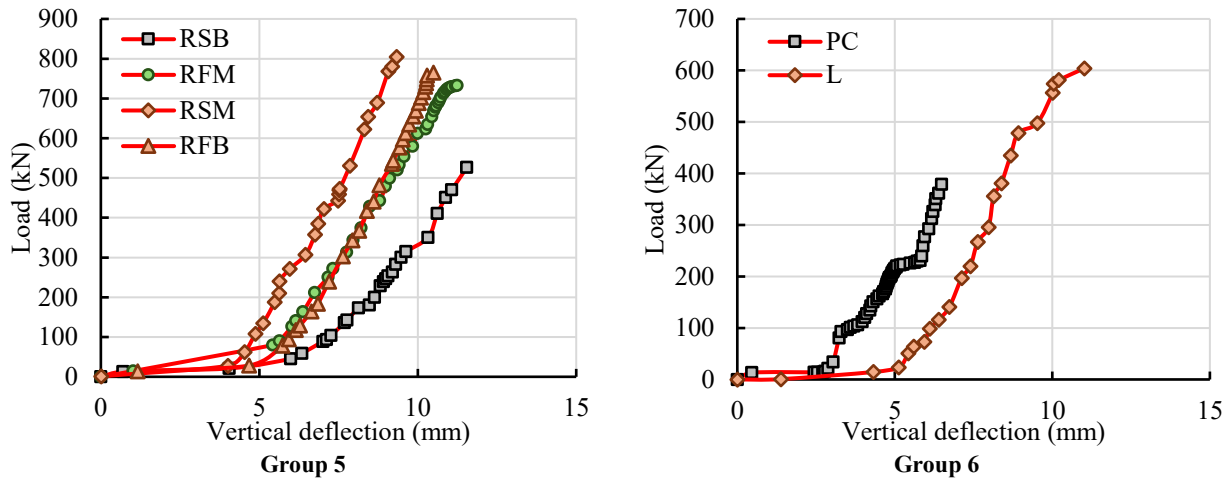


Fig 8. Axial load-deflection response of column groups.

The secant stiffness at 40% of the experimental peak load, $K_{0.4}$, was calculated as in Eq. 2:

$$K_{0.4} = \frac{0.4P_{max}}{\Delta_{0.4}} \quad (2)$$

where P_{max} is the experimental peak axial load and $\Delta_{0.4}$ is the axial displacement corresponding to $0.4P_{max}$. Similarly, the secant stiffness at 80% of the experimental peak load, $K_{0.8}$, was determined as Eq. 3:

$$K_{0.8} = \frac{0.8P_{max}}{\Delta_{0.8}} \quad (3)$$

where $\Delta_{0.8}$ is the axial displacement corresponding to $0.8P_{max}$. The values of $\Delta_{0.4}$ and $\Delta_{0.8}$ were obtained by linear interpolation of the measured ascending branch of the axial load-axial displacement curve. The stiffness values are reported in Table 5.

Table 5. Secant axial stiffness of the tested specimens.

Specimen	P_{max} (kN)	$\Delta_{0.4}$ (mm)	$K_{0.4}$ (kN/mm)	$\Delta_{0.8}$ (mm)	$K_{0.8}$ (kN/mm)
SM	811.37	6.33	51.31	8.51	76.30
SB	705.45	7.54	37.44	9.79	57.64
FM	724.91	7.57	38.30	9.74	59.56
FB	958.48	2.97	129.25	5.96	128.71
LSM	726.67	7.22	40.25	9.86	58.99
LSB	618.76	7.37	33.57	9.26	53.46
LFM	603.87	8.88	27.20	11.28	42.82
LFB	536.56	7.69	27.89	9.65	44.48
SSM	624.59	9.11	27.43	11.42	43.74
SSB	460.83	5.41	34.10	6.54	56.41
SFM	531.38	6.89	30.84	9.35	45.48
SFB	597.40	6.95	34.38	8.74	54.65
HSSM	779.92	7.80	39.97	9.86	63.29
HSSB	607.76	8.02	30.31	9.55	50.93
HSFM	563.10	8.40	26.81	10.52	42.81
HSFB	673.78	8.19	32.90	10.69	50.40
RSM	788.98	6.55	48.20	8.40	75.14
RSB	516.50	8.70	23.73	10.69	38.65
RFM	718.44	7.55	38.07	9.87	58.22
RFB	750.15	7.66	39.16	9.61	62.43
L	592.22	7.51	31.56	9.06	52.27
PC	371.52	4.33	34.35	6.12	48.54

The calculated $K_{0.4}$ values ranged from 23.73 kN/mm for RSB to 129.25 kN/mm for FB, while the $K_{0.8}$ values ranged from 38.65 kN/mm for RSB to 128.71 kN/mm for FB. Specimen FB exhibited the largest secant stiffness at both load levels, consistent with its high peak load and relatively small axial displacement at comparable load levels. In contrast, specimens LFM, LFB, SSM, HSFM, and RSB showed relatively low $K_{0.4}$ values, indicating greater axial deformation before reaching 40% of their respective peak loads.

The stiffness values varied among specimens with similar nominal detailing. Therefore, the observed stiffness response should not be attributed solely to splice type or transverse reinforcement. Differences in concrete compressive strength, local concrete quality, reinforcement location, splice location, and possible residual loading eccentricity may also have influenced the measured response. Because full post-peak descending branches were not consistently recorded, the present study quantifies pre-peak secant stiffness only; post-peak stiffness degradation is discussed qualitatively from the recorded load-displacement curves.

In group 1, the specimens exhibit a steep initial slope and relatively small vertical deflections up to loads of 600-700 kN, reflecting strong confinement and stable elastic behavior; the forged-splice specimen FB shows the highest stiffness, reaching nearly 980 kN at a vertical shortening of only 8 mm. In group 2, where transverse reinforcement was removed from the splice zone, the curves display noticeably larger deflections at comparable loads, with LFM and LFB entering the nonlinear range much earlier and reaching maximum deflections of 10-12 mm at significantly lower loads (500-600 kN). Group 3, incorporating spiral confinement with a 95 mm pitch, showed relatively low pre-peak secant stiffness in several specimens and larger axial displacements at comparable load levels. Because a complete common descending branch was unavailable for all specimens, post-peak stiffness degradation was not quantified. In particular, specimen SSM reached an axial displacement of approximately 12.4 mm, while specimen SFM reached approximately 10.8 mm, both at experimental peak loads below 600 kN. In contrast, specimen SSB reached an axial displacement of approximately 6.9 mm. The larger axial displacements observed in SSM and SFM indicate reduced deformation stability associated with the relatively wide spiral pitch and, potentially, specimen-specific differences in concrete strength and construction quality. In contrast, group 4 demonstrates the beneficial effect of dense spiral reinforcement; specimens like HSSM and HSFM maintain lower deflections throughout loading and reach higher load levels with improved pre-peak secant stiffness and more stable recorded load-displacement responses. However, the response also depended on specimen-specific concrete strength and construction quality; therefore, the effect of reduced spiral pitch should be interpreted as a general trend rather than an isolated variable effect. Square columns in group 5 show intermediate behavior, with RFM and RSM sustaining high loads (700-750 kN) while maintaining controlled deflections around 10-11 mm, reflecting the effectiveness of closed ties in rectangular sections. Finally, the reference columns in group 6 highlight baseline behavior: the plain concrete column PC exhibits a smooth, continuous increase in shortening without a clear peak load, while the continuously reinforced column L shows a more gradual stiffness reduction and reaches deflections above 11 mm at loads around 600 kN. Overall, the vertical load-deflection curves confirm that confinement quality, splice detailing, and concrete integrity strongly influence axial stiffness, deformation capacity, and failure mode across all specimen groups.

3.4. Energy-based ductility index

To quantify the observed differences in deformation capacity and post-peak behavior, an energy-based ductility index, μ_E , was calculated for each specimen from the axial load-axial displacement curves. The index is defined as Eqs. 4 to 6.

$$\mu_E = \frac{E_{total}}{E_{elastic}} \quad (4)$$

while:

$$E_{elastic} = \frac{1}{2} P_{max} \Delta_{max} \quad (5)$$

$$\mu_E = \frac{\int_0^{\Delta_{ref}} P(\Delta) d\Delta}{\frac{1}{2} P_{max} \Delta_{max}} \quad (6)$$

where P_{max} is the experimental peak load and Δ_{max} is the corresponding axial displacement. The reference displacement, Δ_{ref} was defined as the axial displacement at which the load decreased to $0.8P_{max}$ on the descending branch. For specimens that did not exhibit a clear descending branch, Δ_{ref} was taken as the last reliably recorded displacement point.

The calculated energy-based ductility indices are reported in Table 6. Specimens with denser spiral confinement and adequate transverse reinforcement generally exhibited higher μ_E values, reflecting greater energy absorption capacity and more gradual post-peak softening. However, the ductility index also reflects specimen-specific differences in concrete strength and construction quality.

The total absorbed energy ranged from approximately 671 kN·mm for the plain concrete specimen PC to approximately 4320 kN·mm for specimen FB. Specimen FB also exhibited the highest peak load, approximately 958 kN, and the largest normalized energy-absorption ratio, $\mu_E = 1.10$. In contrast, PC showed limited energy absorption because of its lower load capacity and lack of reinforcement. The reinforced control specimen L achieved an intermediate energy absorption of approximately 1925 kN·mm and $\mu_E = 0.59$.

The energy-absorption ratio varied considerably among specimens, ranging from 0.45 to 1.10. For example, LSM and SFM exhibited relatively high values of 0.71 and 0.65, respectively, whereas SSB and HSSB exhibited the lowest values of 0.45. These differences indicate that energy absorption was affected by the combined influence of peak load, axial deformation, splice configuration, transverse confinement, concrete strength, and construction quality. Although some specimens with denser spiral reinforcement showed improved deformation stability, the energy-absorption ratios did not show a uniform trend that could be attributed only to transverse-reinforcement spacing or splice type. Therefore, the energy results should be interpreted together with the peak load, normalized capacity ratio, reinforcement stress, crack width, and observed failure mode.

Table 6. Experimental peak load, axial displacement, absorbed energy, and normalized energy-absorption ratio of the tested reinforced-concrete column specimens.

Specimen	P_{max} (kN)	Δ_{max} (mm)	Δ_{ref} (mm)	E_{total} (kN·mm)	$E_{elastic}$ (kN·mm)	μ_E
SM	811.37	9.84	9.84	2532.99	3989.93	0.63
SB	705.45	10.88	10.88	2105.49	3835.89	0.55
FM	724.91	11.08	11.08	2305.93	4015.99	0.57
FB	958.48	8.21	8.21	4320.11	3936.97	1.10
LSM	726.67	11.40	11.40	2952.59	4142.03	0.71
LSB	618.76	10.48	10.48	1768.24	3240.76	0.55
LFM	603.87	12.64	12.64	2147.72	3817.99	0.56
LFB	536.56	10.46	10.46	1445.46	2807.56	0.51
SSM	624.59	12.40	12.43	2081.25	3873.99	0.54
SSB	460.83	6.88	6.88	709.92	1584.12	0.45
SFM	531.38	10.84	10.84	1872.31	2878.77	0.65
SFB	597.40	9.86	9.86	1632.59	2945.19	0.55
HSSM	779.92	10.86	10.86	2188.25	4233.03	0.52
HSSB	607.76	10.36	10.36	1410.51	3146.66	0.45
HSFM	563.10	11.33	11.33	1602.95	3189.95	0.50
HSFB	673.78	12.10	12.10	2425.30	4078.03	0.59
RSM	788.98	9.34	9.34	2000.21	3684.56	0.54
RSB	516.50	11.55	11.55	1441.66	2982.77	0.48
RFM	718.44	11.25	11.25	2499.18	4041.20	0.62
RFB	750.15	10.50	10.50	1935.69	3938.29	0.49
L	592.22	11.01	11.01	1925.11	3260.19	0.59
PC	371.52	6.48	6.48	670.57	1203.71	0.56

3.5. Comparison with ACI 318 and fib model code provisions

The experimental findings were compared qualitatively with the detailing principles provided in ACI 318-19 [1] and fib model code [28] for reinforced-concrete members containing lap splices. Both design documents recognize that splice performance depends on the bond-transfer capacity between the reinforcement and the surrounding concrete and that transverse reinforcement is important for controlling splitting cracks, restraining lateral dilation, and maintaining the integrity of the concrete surrounding the splice.

ACI 318-19 [1] permits compression lap splices for deformed bars and specifies requirements for lap-splice length, bar size, concrete strength, and transverse reinforcement in column splice regions. For tied columns, ACI 318-19 [1] permits a reduction in compression lap-splice length when ties are provided throughout the lap-splice length in accordance with the relevant code requirements. The results of the present study support the importance of this confinement requirement. The group 2 specimens, in which hoops were omitted over the splice region, generally showed earlier cracking, larger crack widths, and less stable behavior than specimens with transverse reinforcement provided through the splice region. These observations indicate that omission of transverse reinforcement can reduce the confinement effectiveness and increase the vulnerability of the splice region to cracking and localized concrete deterioration.

The fib model code [28] similarly recommends locating splices in regions of relatively low reinforcement stress where possible and emphasizes the role of transverse reinforcement in controlling splitting forces associated with bond transfer. For bars with diameters greater than 12 mm, the fib model code [28] recommends staggering lap splices, where possible, so that no more than half of the reinforcement force is transferred through splices at a single cross-section. It also specifies minimum transverse reinforcement requirements for certain lap-splice conditions and highlights the importance of placing confining reinforcement close to the lapped bars. The present results are consistent with this general design approach, as the specimens with improved spiral confinement or closely spaced transverse reinforcement generally exhibited more stable cracking patterns and improved deformation behavior compared with specimens having insufficient confinement in the splice region.

The results also demonstrate that peak axial load alone is not sufficient to evaluate splice performance. Some forged-splice specimens achieved high peak loads; however, their reinforcement stresses, crack development, and energy-absorption response varied depending on transverse-reinforcement configuration, concrete strength, and fabrication quality. Similarly, lap-splice specimens did not exhibit uniformly superior response under all conditions. Therefore, consistent with the philosophy of both ACI 318 [1] and fib model code [28], the experimental results indicate that splice performance should be evaluated together with concrete quality, transverse confinement, reinforcement stress demand, and the intended structural performance level.

It should be noted that the specimens were tested under monotonic concentric axial compression, whereas code provisions are

intended for a wider range of structural conditions, including members subjected to combined axial load and bending and, in seismic applications, cyclic inelastic deformation. Accordingly, the present findings should not be interpreted as a direct validation or modification of ACI 318-19 [1] or fib model code [28] design provisions. Instead, they provide experimental evidence that supports the general code requirement for adequate transverse reinforcement and careful detailing of splice regions in reinforced-concrete columns.

4. Conclusions

The experimental program conducted in this research involved a comprehensive series of axial compression tests on reinforced-concrete columns containing different splice types, transverse reinforcement configurations, and cross-sectional shapes. The specimens were designed to represent practical construction conditions, including columns with lap splices, flash-butt (forging) splices, individual ties, circular hoops, and spiral reinforcement with varying pitches. The response of the specimens was evaluated using experimental peak load, theoretical capacity, cracking load, axial and lateral displacement, crack width, failure length, local reinforcement stress, concrete surface strain, secant stiffness, and energy absorption. Based on the experimental results, the following conclusions are drawn:

1. Although the columns containing forging (flash-butt welded) splices and those with lap splices exhibited similar behavior in terms of visual and observable performance, the monitoring data obtained from various sensors installed on the longitudinal bars, transverse reinforcement, and concrete indicate meaningful differences. These differences highlight the importance of careful consideration when selecting between the two splice types.
2. Transverse confinement was a major factor influencing the axial response of the tested columns. The specimens with adequate hoops, closed ties, or spiral reinforcement generally exhibited more stable load-displacement behavior, better crack control, and more localized damage than specimens with insufficient confinement in the splice region. The experimental findings confirm that splice performance depends strongly on the quality and continuity of transverse reinforcement.
3. Omitting transverse reinforcement in the splice zone adversely affected the observed response. The group 2 specimens generally showed earlier cracking, wider cracks, and less stable deformation than the specimens with transverse reinforcement maintained through the splice length. The maximum crack width of 3.0 mm was recorded for specimen LSM, which had a lap splice at mid-height without hoops in the splice region. This result emphasizes the importance of transverse reinforcement for controlling splitting cracks and preserving the integrity of the concrete surrounding the splice.
4. Reducing the spiral pitch from 95 mm to 50 mm generally improved confinement effectiveness. The group 4 specimens with a 50 mm spiral pitch showed improved cracking resistance, more stable recorded axial deformation, and generally shorter or more localized failure zones than several group 3 specimens with a 95 mm spiral pitch. Nevertheless, the effect was not identical for every specimen because concrete strength, local concrete quality, splice location, and fabrication conditions also influenced the response.
5. At ultimate load, only one circular column with 95-mm hoop spacing and a bottom lap splice (specimen SB) reached stresses in the longitudinal bars exceeding the yield limit. In this specimen, the concrete strain at failure was approximately 0.0014, which is below the strain limits commonly assumed in design codes (0.003–0.0035). This suggests that columns with longitudinal reinforcement ratios near the upper code limit ($\rho \approx 8\%$) may require further investigation, as the expected ultimate concrete strain may not be achieved uniformly along the height. Additional testing is recommended.
6. The secant stiffness results quantified the pre-peak deformation response of the specimens. The stiffness at 40% of the experimental peak load, $K_{0.4}$, ranged from 23.73 kN/mm for RSB to 129.25 kN/mm for FB. The corresponding stiffness at 80% of peak load, $K_{0.8}$, ranged from 38.65 kN/mm for RSB to 128.71 kN/mm for FB. Specimen FB showed the highest stiffness at both load levels, whereas LFM, LFB, SSM, HFSM, and RSB showed comparatively low pre-peak stiffness. Since a complete and consistent post-peak descending branch was unavailable for all specimens, post-peak stiffness degradation was not quantified consistently.
7. The capacity results should be interpreted together with the measured concrete strength. The concrete compressive strength varied among the specimens; consequently, direct comparison based on peak load alone does not isolate the effect of splice type or confinement detail. The experimental-to-theoretical capacity ratio, together with stiffness, crack width, deformation, reinforcement stress, and energy absorption, provides a more reliable basis for comparing specimen performance.
8. Neither lap splices nor flash-butt-forged splices showed universally superior performance in all test configurations. Some forged-splice specimens, particularly FB, achieved high peak load, high secant stiffness, and high energy absorption. However, other forged-splice specimens showed lower capacity or less stable deformation. Likewise, lap-splice specimens did not consistently provide higher peak capacity than forged-splice specimens. The splice response was governed by the combined effects of splice type, splice location, transverse confinement, concrete quality, and fabrication quality.
9. The reduced longitudinal bar stress in specimen RFM compared to RSM, despite neither specimen reaching yield in the longitudinal bars, is explained in part by differences in transverse reinforcement behavior and concrete strength. The ties in both specimens exceeded yield, reaching 425 MPa in RFM and 415 MPa in RSM. Although RSM carried approximately

2.5% more than its theoretical ultimate load, RFM experienced a 12% reduction relative to its theoretical capacity. These differences highlight the sensitivity of forged-splice specimens to transverse-reinforcement demand and concrete quality. In this specific comparison, the lap-splice specimen exhibited a more favorable combination of capacity and reinforcement stress, but this trend should not be generalized to all forged- and lap-splice configurations without considering confinement, concrete strength, and fabrication quality.

10. At ultimate load, transverse reinforcement stresses in many forged-splice specimens were higher than in comparable lap-splice specimens, except in specimens SFM and LFB. The lower transverse steel stress in several lap-splice columns indicates a reduced demand on the confinement reinforcement under axial compression, which can be beneficial for ductility and post-peak stability. However, this trend was not uniform across all specimens and should be interpreted together with the effects of confinement configuration, spacing, and concrete strength.
11. Cracking in all columns (square and circular, with any transverse reinforcement type or spacing) containing either forging or lap splices occurred after the service load, except in group 2. Thus, both splice types can safely sustain service loads without cracking. The premature cracking in group 2 is attributed to the absence of transverse reinforcement over the 300-mm splice region.
12. The cracking observations show that both lap-splice and forged-splice columns generally remained uncracked under the stated service-load level when adequate transverse reinforcement was provided. In contrast, the specimens without transverse reinforcement in the splice region showed premature cracking. This result confirms that confinement detailing in the splice zone is essential for serviceability as well as ultimate-response performance.
13. The energy-based results indicate substantial variation in the deformation and load-carrying response among specimens. The normalized energy-absorption ratio, μ_E , ranged from 0.45 to 1.10. Specimen FB exhibited the largest value, whereas SSB and HSSB exhibited the lowest values. The results show that energy absorption was affected by the combined influence of load capacity, axial deformation, confinement arrangement, splice detailing, concrete strength, and construction quality. Therefore, the energy-based index should be considered together with the other measured response parameters.
14. The conclusions of this study are limited to short columns subjected to monotonic concentric axial compression. The behavior under combined axial load and bending, cyclic seismic loading, fatigue, repeated loading, and full-scale structural conditions was not investigated. In addition, variation in concrete strength, possible local construction defects, and the absence of direct measurement of initial loading eccentricity contributed to specimen-to-specimen scatter. Further tests with repeated specimens, controlled material properties, documented splice-quality testing, and cyclic axial-flexural loading are recommended.

Statements & declarations

Author contributions

Mohammad Maghsoudi: Conceptualization, Methodology, Validation, Visualization, Investigation, Writing – Original Draft & Editing.

Amirreza Toghrol Pour: Writing - Review & Editing, Visualization, Data curation, Investigation.

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Data availability

The experimental data supporting the findings of this study are available from the corresponding author upon reasonable request.

Declarations

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.

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A Technology-Maturity-Based Foresight Portfolio Framework for Smart Seismic Structures: Integrating CRITIC, SWARA-Based Weighting, Fuzzy VIKOR, Borda Analysis, and Monte Carlo Uncertainty Assessment

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ABSTRACT

The rapid development of smart seismic technologies requires structured approaches for selecting foresight methods according to technology maturity. Existing studies mainly address the technical development or performance assessment of individual technologies, while limited attention has been given to maturity-sensitive foresight portfolio design. This study proposes a technology-maturity-based foresight portfolio framework for smart seismic structures. First, scientific and patent-based evidence is screened and coded. The CRITIC method is used to weight evidence indicators and construct weighted maturity profiles, after which technologies are assigned to four planning-oriented maturity/application stages using predefined profile-based classification rules: emerging/research-oriented, developing/technology-readiness, implementation-oriented, and recovery/resilience-oriented. Second, twelve valid expert questionnaires are used to evaluate decision criteria and foresight methods. The criteria are weighted through a SWARA-based procedure, and eight foresight methods are ranked using Fuzzy VIKOR. Third, the overall VIKOR ranking is complemented by a separate tied-rank Borda analysis based on experts' stage-specific portfolio assessments to extract maturity-sensitive foresight portfolios. Robustness is examined using expert agreement and reliability checks, Monte Carlo uncertainty analysis, VIKOR sensitivity analysis, and comparative validation with TOPSIS and PROMETHEE II. The results show that seismic resilience support is the most important criterion. Delphi and technology roadmapping form the most stable overall foresight core, while stage-specific portfolios vary across maturity contexts. The proposed framework supports maturity-sensitive foresight planning for resilient smart seismic infrastructure.

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1. Introduction

Earthquakes remain among the most destructive hazards affecting buildings, bridges, lifeline systems, and critical infrastructure. In modern earthquake engineering, infrastructure performance is no longer evaluated only in terms of life safety or collapse prevention. Increasing attention is now given to resilience-oriented objectives, including damage limitation, residual drift control, reparability, reduced downtime, re-occupancy, functional recovery, and lifecycle seismic loss. These objectives require structural systems that can not only resist seismic actions, but also support damage detection, post-earthquake assessment, adaptive

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maintenance, and rapid recovery [1].

Smart earthquake-proofing structures are a significant milestone in the resilient paradigm. Such systems feature seismic-proofing technology combined with the ability to monitor, analyze, control, and make decisions based on collected data. Examples of such systems include structural health monitoring systems, smart sensors, IoT seismic monitoring, semi-active control systems, magnetorheological dampers, self-centering systems, base isolation, supplementary damping systems, replaceable fuses, post-earthquake rapid damage assessment systems, and digital twin-based systems. Improvements in IoT-based structural health monitoring and digital twins show that smart infrastructure systems have the potential to improve condition assessment and decision-making [2, 3]. Structural damage detection has emerged as a fundamental component of resilient and intelligent infrastructure systems, as it facilitates the timely identification, localization, and assessment of structural deficiencies, thereby enhancing post-event safety evaluation, informing maintenance strategies, and supporting rapid functional recovery [4, 5].

However, smart seismic technologies do not have the same level of maturity or implementation readiness. Some technologies, such as AI-based damage detection and digital twins, remain strongly research-oriented in many seismic applications. Others, such as structural health monitoring, smart sensors, IoT-based monitoring, and semi-active control devices, show increasing technology-readiness evidence. More established technologies, such as base isolation, supplemental damping devices, and self-centering systems, are closer to engineering implementation. Recovery-oriented solutions, including replaceable fuse systems, rapid damage assessment, and re-occupancy or downtime-reduction tools, are increasingly important for post-earthquake resilience planning. Therefore, planning for smart seismic structures requires not only identifying promising technologies, but also understanding which foresight methods are most appropriate for different maturity contexts.

Technology foresight methods can support this decision-making process by helping engineers, infrastructure owners, and policymakers identify emerging trends, assess technology readiness, evaluate implementation barriers, and plan development pathways. Different foresight methods provide different types of knowledge. Bibliometrics can identify scientific trends and research clusters. Patent analysis can reveal innovation activity, technological maturity, and commercialization signals. Horizon scanning can detect weak signals and early-stage opportunities. Delphi can support structured expert consensus under uncertainty. MACTOR can help analyze stakeholder interests and implementation feasibility. Scenario planning can explore alternative future conditions, while technology roadmapping can translate foresight results into staged implementation pathways. These methods are therefore complementary rather than interchangeable [6, 7].

For this reason, selecting a single “best” foresight method may not be sufficient for smart seismic technology planning. A method that is effective for detecting emerging research trends may not be adequate for assessing implementation feasibility or post-earthquake recovery needs. Similarly, a method suitable for mature engineering technologies may not be appropriate for early-stage digital or data-driven solutions. This suggests that foresight-method selection should be linked to technology maturity. In other words, smart seismic technologies may require maturity-sensitive portfolios of foresight methods instead of a universal single-method solution. To address this need, this study proposes a technology-maturity-based foresight portfolio framework for smart seismic structures. The framework first screens and codes scientific and patent-based evidence and applies the CRITIC method to classify smart seismic technologies into maturity/application stages. Then, expert-based multi-criteria decision analysis is used to evaluate foresight methods with respect to criteria that reflect both foresight capability and seismic engineering relevance. The criteria are weighted through a SWARA-based procedure, and the foresight methods are ranked using Fuzzy VIKOR. A tied-rank Borda analysis is then applied to extract suitable foresight portfolios for each maturity stage. Finally, Monte Carlo uncertainty analysis, VIKOR sensitivity analysis, and comparative validation with TOPSIS and PROMETHEE II are used to examine the robustness of the results.

The proposed framework will consider eight different methods of foresight, including: brainstorming, MACTOR/stakeholder analysis, bibliometrics, patent analysis, Delphi, horizon scanning, scenario planning, and technology roadmapping. Criteria for the decision-making include identification of new scientific and technological trends, weak signal detection, assessment of the technology maturity/ready state, coherence with the objective of seismic resilience, relevance to smart seismic technologies, stakeholder acceptability, long-term uncertainty management, and assistance in planning implementation. Altogether, these criteria allow the framework to connect foresight-method selection with issues relevant for earthquake engineering.

The main contribution of this study is the development of an integrated decision-support framework that links technology maturity, foresight portfolio selection, seismic resilience, and uncertainty analysis. Unlike conventional ranking models that identify one preferred foresight method, the proposed framework determines which combination of methods is more suitable for each maturity stage of smart seismic technologies. This maturity-sensitive perspective helps transform foresight from a generic technology-planning tool into a practical decision-support approach for resilient smart seismic infrastructure.

The remainder of this paper is organized as follows. Section 2 reviews the related work on smart seismic technologies, seismic resilience, technology maturity, foresight methods, and uncertainty-aware decision support. Section 3 presents the research gap and contributions of the study. Section 4 describes the methodology, including CRITIC-based maturity assessment, expert questionnaire design, SWARA-based criteria weighting, Fuzzy VIKOR ranking, tied-rank Borda portfolio extraction, and robustness validation. Section 5 presents and discusses the results, including maturity classification, criteria weights, foresight-method rankings, portfolio recommendations, and practical decision guidelines. Section 6 concludes the paper, and Section 7 discusses limitations and future research directions.

2. Related work

This section reviews the literature in three interrelated areas. First, it examines smart seismic technologies and their contribution to resilience-oriented earthquake engineering. Second, it discusses how differences in technological maturity, evidence availability, and implementation readiness create different planning and foresight requirements. Third, it reviews the main foresight methods and uncertainty-aware decision-support approaches relevant to the selection of maturity-sensitive foresight portfolios. The review demonstrates that smart seismic technologies should not be treated as a homogeneous technological domain and that foresight methods should be selected according to the maturity, application context, and strategic purpose of the technologies under consideration.

2.1. Smart seismic technologies and resilience-oriented applications

Resilience to earthquakes has become one of the key indicators in the assessment of structural and urban resilience [8]. It is the ability of the system to maintain its essential function and recover from earthquake effects. Earthquake engineering has gradually moved beyond the traditional objectives of strength, ductility, life safety, and collapse prevention toward a broader resilience-oriented perspective. In this perspective, acceptable seismic performance also includes damage limitation, reparability, residual-drift control, reduced downtime, rapid re-occupancy, functional recovery, and the restoration of infrastructure services. Consequently, the performance of buildings and infrastructure is increasingly evaluated not only by their response during an earthquake, but also by the time, resources, and interventions required to restore their intended functionality after the event [9–11]. The NIST-FEMA functional-recovery framework similarly identifies re-occupancy and recovery time as explicit performance objectives for buildings and lifeline systems. Bitarafan et al. [12] studied the importance of selecting appropriate smart structural design scenarios to enhance resilience-oriented seismic performance under future earthquake conditions.

Seismic technologies help achieve these goals by ensuring that structural engineering systems are combined with the technologies of sensing, monitoring, communication, control, data analysis, and engineering decision-making. Major technologies include Structural Health Monitoring (SHM), smart sensors, wireless sensor networks, IoT-based seismic monitoring, and edge computing technology. These technologies provide continuous or event-triggered data about the structural response and condition, allowing one to detect any anomalies, post-earthquake evaluation, schedule maintenance, and rapid engineering decision-making. IoT-based SHM helps combine sensors, communication infrastructure, and data-processing capabilities, while edge computing helps avoid the necessity for full centralization of the process and allows real-time analysis at the site of monitoring [2, 13].

Artificial-intelligence-based damage detection represents a closely related but rapidly developing research direction. Machine-learning and deep-learning models can analyse vibration signals, images, and other monitoring data to identify damage-sensitive patterns that may be difficult to detect through conventional approaches. However, although these methods have demonstrated strong performance in laboratory experiments, benchmark datasets, and numerical applications, their routine use in seismic engineering still depends on the availability of representative training data, model interpretability, uncertainty quantification, and validation under real operating and earthquake conditions [14].

Digital twins extend monitoring and data-driven assessment by linking physical infrastructure to dynamically updated virtual representations. In civil infrastructure, digital twins may combine sensor measurements, inspection records, numerical models, maintenance information, and performance simulations to support condition assessment and lifecycle decision-making. Nevertheless, the literature also identifies unresolved challenges related to semantic interoperability, integration of heterogeneous data, sensor-network representation, model updating, and synchronization between physical and digital systems [3, 15]. Recent bridge applications indicate that digital-twin-based SHM is progressing toward field implementation, although many applications remain project-specific and require broader standardization and validation [16]. A second group of smart seismic technologies is associated with structural response control and seismic protection. This group includes semi-active control systems, magnetorheological dampers, adaptive control strategies, base-isolation systems, and supplemental energy-dissipation devices. Semi-active systems can modify their mechanical properties in response to measured structural behaviour while generally requiring less external energy than fully active systems. Their potential has been demonstrated in structural-control research and full-scale applications, but practical adoption requires attention to reliability, control stability, power availability, maintenance, sensor performance, and fail-safe behaviour [17].

Self-centering and low-damage structural systems provide another important pathway toward seismic resilience. Self-centering systems combine recentering capability with energy dissipation to reduce permanent structural deformation following earthquakes. Reduced residual drift can decrease the probability that an otherwise stable structure is judged economically irreparable and can facilitate inspection, repair, and return to service [18]. Similarly, replaceable fuse mechanisms are intended to concentrate inelastic damage in predefined components that can be inspected and replaced more efficiently than primary structural members. These systems therefore connect structural-response reduction with reparability and post-earthquake recovery objectives [19].

Recent research has increasingly formalized the assessment of downtime and functional recovery. Molina Hutt et al. [20] proposed a probabilistic framework that distinguishes recovery states such as stability, shelter-in-place, re-occupancy, functional recovery, and full recovery. Cook et al. [21] further operationalized functional-recovery assessment by linking component damage states to system-level operation and building-level re-occupancy and functionality. These studies show that recovery is influenced not only by structural damage, but also by damage to non-structural components, utilities, inspection procedures, repair sequencing, labour availability, and other impeding factors. The uncertainty associated with these variables is itself a major decision concern. Kourehpaz et al. [22] demonstrated that assumptions regarding seismic demands, component fragilities, replacement consequences,

and impeding-factor delays can substantially influence loss and recovery-time estimates. More recent downtime-oriented studies have also argued that future seismic design objectives should explicitly address functional disruption in addition to life safety and collapse prevention [23]. Therefore, recovery-oriented smart technologies should be assessed not only in terms of their technical capability, but also according to their potential contribution to reducing inspection time, repair delay, re-occupancy time, and overall functional disruption. Recent applied studies have also highlighted the importance of resilience-oriented assessment and mitigation planning in infrastructure systems under hazard-induced disruption, further emphasizing the need for context-sensitive planning and recovery-oriented decision-making in seismic applications [24, 25].

Overall, smart seismic technologies can be organized into three complementary functional domains. The first is monitoring and diagnosis, including SHM, IoT-based monitoring, smart sensors, AI-based damage detection, and digital twins. The second is response control and damage reduction, including base isolation, supplemental damping, semi-active control, magnetorheological dampers, and self-centering systems. The third is repairability and recovery support, including replaceable components, rapid post-earthquake assessment, re-occupancy decision support, and downtime-reduction systems. Although these domains are interdependent, the technologies within them differ substantially in research maturity, validation evidence, field implementation, standardization, and recovery relevance. These differences provide the first justification for adopting a maturity-sensitive foresight framework.

2.2. Technology maturity and foresight needs in smart seismic structures

Technology maturity is commonly represented using Technology Readiness Levels or similar stage-based models. Such models are useful for distinguishing conceptual research, laboratory validation, prototype development, and operational deployment. However, maturity cannot always be represented adequately by a single linear scale. Previous research has identified limitations related to system complexity, the validity of readiness assessments, integration between technological components, and the use of readiness levels for planning and review [26]. Accordingly, the maturity categories used for smart seismic technologies should be interpreted as planning-oriented categories rather than formal or universal TRL assignments.

For smart seismic technologies, maturity is multidimensional. Scientific publications indicate research visibility, knowledge development, and experimental validation, while patents can provide complementary information regarding invention activity, technological embodiment, and potential commercialization. Bibliometric and patent information are particularly valuable when historical market data are limited, as is often the case for emerging technologies. Daim et al. [27] demonstrated that combining bibliometric and patent evidence with forecasting approaches can strengthen the analysis of emerging technology trajectories. However, publication or patent activity alone should not be equated with implementation readiness; field validation, engineering standards, interoperability, maintainability, stakeholder acceptance, and recovery contribution must also be considered.

AI-based damage detection and digital twins may be regarded as emerging or research-oriented in many seismic applications. Their scientific visibility has increased rapidly, and their potential for automated diagnosis, predictive maintenance, and lifecycle management is substantial. Nevertheless, many studies continue to rely on laboratory data, numerical simulations, benchmark structures, or project-specific implementations. Their transition into routine seismic design and post-earthquake practice requires reliable data pipelines, validation under changing environmental and operational conditions, interpretable models, professional acceptance, and compatibility with existing engineering procedures [3, 14, 15].

SHM platforms, smart sensors, IoT-based monitoring, edge-computing applications, and semi-active control devices generally represent a developing or technology-readiness-oriented group. These technologies possess stronger experimental, prototype, and application evidence than many fully data-driven solutions. Nevertheless, their readiness differs across structural types and implementation environments. Sensor durability, communication reliability, power requirements, cybersecurity, data management, maintenance, and compatibility with engineering decision processes can influence whether a technically validated system can be adopted at scale [2, 13, 17].

Base isolation, supplemental damping devices, and several self-centering systems are more closely associated with implementation-oriented maturity. These technologies have longer analytical and experimental histories and clearer connections to design practice and structural-performance objectives. However, their implementation still depends on code provisions, constructability, project cost, procurement practices, inspection requirements, maintenance responsibilities, and stakeholder confidence. Consequently, foresight for relatively mature technologies should focus less on detecting scientific novelty and more on understanding adoption barriers, regulatory pathways, market conditions, and staged deployment. Recovery – and resilience – oriented technologies create a different maturity context. Replaceable fuse systems, rapid damage-assessment tools, re-occupancy decision platforms, and downtime-reduction systems are not necessarily less technologically developed than implementation-oriented devices. Instead, they are distinguished by their principal application objective: the restoration of functionality after earthquakes. Their assessment therefore requires evidence related to repair sequencing, inspection, workforce and resource availability, building services, non-structural damage, operational continuity, and acceptable recovery times [11, 20, 21].

These maturity differences create distinct foresight requirements. For emerging technologies, foresight should emphasize scientific mapping, weak-signal detection, research-gap identification, and expert interpretation. For developing technologies, it should incorporate patents, validation evidence, readiness constraints, and pathways from research to application. For implementation-oriented technologies, stakeholder interests, regulation, costs, institutional conditions, and deployment planning become more influential. For recovery-oriented technologies, foresight must additionally consider uncertain damage and recovery scenarios, service-restoration targets, resource constraints, and long-term resilience expectations. The selection of foresight methods

should therefore be related to both technological maturity and the intended decision outcome. Research on foresight-method selection indicates that methods are often chosen on the basis of familiarity or intuition, even though their suitability depends on objectives, available evidence, participation requirements, resources, and uncertainty conditions [28]. Similarly, bibliometric and patent evidence can support the assessment of emerging and developing technologies, but they should be combined with qualitative and participatory methods when implementation, stakeholders, and future uncertainty are important [27].

Accordingly, technology maturity provides a defensible basis for designing foresight portfolios. It does not imply that each technology follows a single fixed development trajectory or that one method corresponds exclusively to one maturity stage. Instead, maturity indicates the dominant types of evidence, uncertainty, actors, and planning questions that the foresight process must address. This interpretation supports the use of complementary portfolios rather than a universal foresight method.

2.3. *Foresight methods and uncertainty-aware decision support*

Technology foresight provides structured approaches for anticipating technological change, identifying opportunities and risks, and developing strategic responses under uncertainty. The literature includes a wide range of exploratory, evidential, expert-based, participatory, and planning-oriented methods. Because these methods generate different forms of knowledge, their selection should depend on the problem context, the available information, the maturity of the technology, and the intended use of the foresight outputs. Popper [28] showed that foresight-method selection is itself a methodological problem, while more recent taxonomies confirm the diversity of foresight techniques and the need to align methods with strategic objectives [8, 29].

Bibliometrics is particularly useful for mapping scientific growth, research clusters, influential publications, collaboration networks, and emerging topics. Patent analysis provides a different but complementary view by identifying invention activity, technological claims, assignees, application areas, and possible commercialization signals. The integration of these evidence streams can improve the analysis of emerging technologies, especially when reliable market information is unavailable [27]. For smart seismic technologies, these methods can help distinguish rapidly expanding research areas from technologies showing stronger device-level or application-oriented development.

Delphi is a structured expert-elicitation method that uses iterative questionnaires, controlled feedback, and typically anonymous participation to develop judgments under uncertainty. Its usefulness is particularly strong when empirical evidence is incomplete or when future development depends on multidisciplinary expertise. However, its credibility depends on appropriate expert selection, questionnaire design, feedback procedures, and transparent definitions of agreement or consensus [30]. Delphi is therefore well suited to interpreting uncertain maturity evidence and evaluating the feasibility and resilience contribution of smart seismic technologies.

Horizon scanning focuses on identifying early signals, emerging developments, discontinuities, and potential risks or opportunities. It is particularly relevant for early-stage digital and data-driven technologies whose future applications may not yet be visible in established engineering practice. Recent OECD guidance emphasizes that horizon scanning can support anticipatory technology policy, but also notes challenges related to signal definition, evidence integration, methodological quality, and the translation of scanning results into usable decisions [31].

Scenario planning is used to explore multiple plausible futures rather than predict a single expected outcome. It can incorporate changes in hazard conditions, technology development, regulation, investment, stakeholder expectations, and recovery requirements. Reviews of scenario-planning approaches show that combinations of qualitative and quantitative techniques can improve scenario development, selection, and validation [32]. In smart seismic planning, scenario planning is especially relevant where recovery objectives, resource availability, damage conditions, and acceptable downtime remain uncertain.

MACTOR and related stakeholder-analysis methods focus on actors, objectives, influence relationships, cooperation, and conflict. Such methods become increasingly important when technological performance is no longer the main barrier and implementation depends on infrastructure owners, engineers, contractors, regulators, technology suppliers, insurers, occupants, and emergency-management organizations. Stakeholder analysis can therefore complement technical assessment by identifying institutional constraints, adoption incentives, and potential conflicts affecting implementation. Technology roadmapping links markets, needs, products, systems, technologies, resources, and time horizons within a structured development pathway. It is widely used to support long-term strategic planning and to communicate relationships between evolving needs and technological capabilities [33]. Roadmapping is especially valuable for developing and implementing smart seismic technologies because it can convert scientific, patent, expert, and stakeholder evidence into milestones, responsibilities, validation requirements, and deployment sequences. Brainstorming can provide rapid early-stage idea generation, but its outputs are generally less structured and more dependent on group composition and facilitation. It is therefore more appropriate as a supporting method than as a complete foresight framework. Similarly, no single method can simultaneously provide scientific mapping, weak-signal detection, expert consensus, stakeholder analysis, scenario exploration, and implementation planning. This supports a portfolio perspective in which complementary methods are combined according to the maturity stage and decision need.

The selection and ranking of foresight methods are also affected by uncertainty. Expert judgments are frequently expressed through linguistic assessments, evidence concerning emerging technologies may be incomplete, and alternative criteria may conflict. Fuzzy logic can represent the vagueness of linguistic judgments, while multi-criteria methods can integrate technical, resilience, implementation, stakeholder, and uncertainty-related criteria. SWARA supports stepwise expert weighting, and VIKOR identifies compromise alternatives by balancing overall utility and maximum individual regret. Sensitivity analysis, Monte Carlo simulation,

TOPSIS, and PROMETHEE II can then be used to examine whether the resulting rankings remain stable under alternative assumptions [34–38].

Taken together, the literature provides strong technical knowledge regarding individual smart seismic technologies and a broad range of foresight and decision-support methods. However, the two research streams remain weakly integrated. Existing smart seismic studies primarily examine structural performance, monitoring, control, damage assessment, or recovery, whereas foresight studies generally discuss method selection independently of earthquake-engineering applications. The literature therefore does not clearly explain how foresight methods should be combined for smart seismic technologies at different levels of maturity and application readiness. This gap provides the basis for the maturity-sensitive foresight portfolio framework proposed in this study.

3. Research gap and contributions

Previous studies on intelligent seismic structures have focused mainly on the technological innovation and performance analysis of various individual technologies used, including structural health monitoring technology, smart sensors, artificial intelligence-based damage detection, digital twin technology, semi-active control technology, self-centering technology, replaceable fuses technology, and earthquake rehabilitation technologies. All these studies give an insight into how each of these technologies can be employed to reduce the seismic effects, increase monitoring, help in damage detection, and restore functionality. However, these technologies have been considered mainly from a technological perspective, rather than with strategic considerations related to their maturity stages.

In parallel, the technology foresight literature offers several methods for supporting long-term decision-making under uncertainty, such as Delphi, horizon scanning, bibliometrics, patent analysis, scenario planning, stakeholder analysis, and technology roadmapping. Recent studies emphasize that foresight methods should be selected according to the decision context, strategic purpose, and maturity of the technology rather than applied as fixed universal tools [29, 39]. Nevertheless, the direct use of foresight for smart seismic technologies remains limited. More specifically, there is still no clear framework that explains which foresight methods are more suitable for emerging, developing, implementation-oriented, and recovery-oriented smart seismic technologies. This gap is important because each maturity stage creates a different planning need. Emerging technologies require scientific trend analysis and weak-signal detection; developing technologies require readiness assessment, patent evidence, and transition planning; implementation-oriented technologies require stakeholder analysis, feasibility assessment, and deployment pathways; and recovery-oriented technologies require attention to repairability, downtime, re-occupancy, and functional recovery. Recent seismic recovery studies further confirm that functional recovery and re-occupancy should be treated as explicit objectives in resilience-oriented planning rather than secondary outcomes [40].

Therefore, the main gap addressed in this study is the lack of a technology-maturity-based foresight portfolio framework for smart seismic structures. Existing research provides strong technical knowledge on individual smart seismic technologies, and foresight research provides useful planning methods. However, these two streams have not been sufficiently integrated into a decision-support framework that links technology maturity with suitable combinations of foresight methods. To the best of the authors' knowledge, no prior study has developed such a maturity-based foresight portfolio framework specifically for smart seismic structures. To address this gap, this study proposes a maturity-sensitive foresight portfolio framework. Instead of identifying a single best foresight method, the framework treats foresight-method selection as a portfolio design problem. The main contributions of the study are as follows: first, it classifies smart seismic technologies into maturity/application stages using scientific and patent-based evidence; second, it evaluates foresight methods through an SWARA-based weighting and Fuzzy VIKOR decision model; third, it identifies suitable foresight portfolios for each maturity stage; and fourth, it tests the robustness of the results using Monte Carlo uncertainty analysis, VIKOR sensitivity analysis, and comparative validation with alternative multi-criteria decision-making methods. Through these contributions, the study provides a practical decision-support tool for planning resilient smart seismic infrastructure.

4. Methodology

This study proposes a maturity-based decision-support framework for selecting foresight portfolios for smart seismic technologies. The framework combines evidence-based maturity assessment, expert-based multi-criteria evaluation, portfolio extraction, and robustness validation. The main objective is to match different smart seismic technology stages with suitable combinations of foresight methods, rather than selecting a single best method for all technologies.

The process begins with the selection of smart seismic technologies that represent the case study and the collection of scientific and patent evidence. The collected evidence is then used to evaluate the level of maturity and application of each selected technology. The weights of the evidential indicators are obtained using the CRITIC method and then used to generate weighted maturity profiles. In the next step, the technologies are categorized into four levels of maturity and application.

In the next stage, decision criteria and foresight methods are defined based on the literature review and twelve valid expert questionnaires. The criteria cover trend identification, weak-signal detection, technology readiness, applicability to smart seismic technologies, seismic resilience support, stakeholder acceptance, long-term uncertainty handling, and implementation planning. Since expert evaluations are expressed through linguistic judgments, fuzzy numbers are used to model the vagueness of the assessments. The criteria are weighted using a SWARA-based procedure, and the foresight methods are ranked using Fuzzy VIKOR. SWARA supports stepwise criterion weighting based on expert judgment, while VIKOR identifies compromise priorities among alternatives under multiple and potentially conflicting criteria [35, 37].

The overall VIKOR ranking was complemented by a separate tied-rank Borda analysis based on experts' stage-specific portfolio assessments. While Fuzzy VIKOR identified the overall compromise priority of foresight methods across the decision criteria, tied-rank Borda analysis was used to extract maturity-sensitive foresight portfolios for each technology stage.

Finally, the robustness of the results is examined through Monte Carlo uncertainty analysis, VIKOR sensitivity analysis, and comparative validation with TOPSIS and PROMETHEE II. This validation stage evaluates whether the proposed rankings and portfolios remain stable under input uncertainty, changes in the VIKOR compromise coefficient, and alternative multi-criteria decision-making methods. The final output is a set of maturity-sensitive foresight portfolios and practical decision guidelines for planning resilient smart seismic infrastructure.

The methodological workflow of the proposed framework is shown in Fig. 1. The process begins with technology identification and evidence collection, followed by CRITIC-based maturity classification. Expert questionnaires are then used to define decision criteria, weight the criteria through SWARA, and rank foresight methods using Fuzzy VIKOR. The overall Fuzzy VIKOR ranking is complemented by a separate tied-rank Borda analysis based on experts' stage-specific portfolio assessments. The Borda analysis is then used to extract maturity-sensitive foresight portfolios for each technology stage. Finally, Monte Carlo analysis, VIKOR sensitivity analysis, and TOPSIS/PROMETHEE II validation are applied to examine the robustness of the results.



Fig. 1. Methodological workflow of the proposed maturity-based foresight portfolio framework for smart seismic technologies.

4.1. Evidence-based maturity assessment of smart seismic technologies

This section explains how the maturity/application stages of smart seismic technologies were assessed before selecting foresight portfolios. The purpose of this step was to create a transparent and comparable maturity profile for the selected technologies. The objective was not to conduct a full systematic review, bibliometric analysis, or comprehensive patent-landscaping study. Instead, a structured scientific and patent-based evidence screening was used to support ordinal maturity coding for the CRITIC decision matrix. First, ten representative smart seismic technology groups were identified based on the literature review and the scope of the study. These groups included AI-based damage detection, digital twins, structural health monitoring and advanced smart sensing, smart sensors and IoT-based seismic monitoring, magnetorheological dampers and semi-active control, self-centering systems, base isolation systems, supplemental damping devices, replaceable fuse and low-damage systems, and rapid damage assessment or re-occupancy support tools. These technologies were selected because they represent different functions of smart seismic structures, including monitoring and diagnosis, response control, implementation-oriented seismic protection, and post-earthquake recovery support.

To improve the transparency and reproducibility of the CRITIC-based maturity assessment, a structured evidence screening protocol was applied before constructing the coded maturity matrix. The screening was conducted at the technology-group level. For each technology group, the first twenty most relevant scientific records and the first ten most relevant patent records were screened. Therefore, the evidence base included 200 scientific records and 100 patent records in total.

The scientific sources were selected based on searches around the topic on Google Scholar, while the patent-based sources were selected using Google Patents. The searches were done from the year 1990 to 2026, the latest search being done in July 2026. Keywords related to the technology along with keywords related to seismic and earthquake engineering, structural systems, civil structures, monitoring, sensing, damage detection, response control, damping, isolation, recovery, re-occupancy, and resilience were used. Records were included when they were directly related to seismic, earthquake, structural, civil infrastructure, monitoring, sensing, damage detection, response control, damping, isolation, post-earthquake assessment, reparability, re-occupancy, recovery, or resilience applications. Records were excluded when they were duplicated, unrelated to civil or seismic engineering, focused only on non-structural applications, or did not have a clear connection to smart seismic technologies. The screening protocol is summarized in Table 1.

Table 1. Evidence screening protocol for CRITIC-based maturity assessment.

Item	Description
Scientific evidence source	Google Scholar-oriented search and indexed publisher records
Patent evidence source	Google Patents
Search period	1990-2026
Final search date	July 2026
Number of technology groups	10
Scientific records screened per group	20 records
Patent records screened per group	10 records
Total scientific records screened	200 records
Total patent records screened	100 records
Total evidence records screened	300 records
Screening approach	Structured evidence screening for ordinal maturity coding
Purpose	To support CRITIC-based maturity assessment, not to conduct a full systematic review, bibliometric analysis, or patent-landscaping study
Inclusion criteria	Records related to seismic, earthquake, structural, civil infrastructure, monitoring, sensing, damage detection, response control, damping, isolation, post-earthquake assessment, reparability, recovery, re-occupancy, or resilience
Exclusion criteria	Duplicate records, unrelated non-civil applications, non-structural records, and records without clear relevance to smart seismic technologies
Coding scale	Low = 1 ; low-medium = 2 ; medium = 3 ; medium-high = 4 ; high = 5 ; very high = 6

Table 2 shows the screening depth for each technology cluster. The organization helped ensure that the same amount of scientific and patent literature was taken into consideration for all the groups regardless of their varying degrees of visibility.

Table 2. Screening depth by technology group.

Technology group	Scientific records screened	Patent records screened	Total records screened
AI-based damage detection	20	10	30
Digital twins	20	10	30
SHM/advanced smart sensing	20	10	30
Smart sensors/IoT-based seismic monitoring	20	10	30
MR dampers/semi-active control	20	10	30
Self-centering systems	20	10	30
Base isolation systems	20	10	30
Supplemental damping devices	20	10	30
Replaceable fuse/low-damage systems	20	10	30
Rapid damage assessment/re-occupancy tools	20	10	30
Total	200	100	300

After screening, the retained evidence was coded using ten maturity-related indicators: five scientific indicators and five patent-based indicators. The scientific indicators captured research visibility, recent research growth, experimental or validation evidence, implementation or case-study evidence, and recovery/resilience relevance. The patent-based indicators captured patent visibility, seismic/civil patent relevance, device or system maturity, recent patent activity, and recovery/resilience patent signals. These indicators are presented in Table 3.

Table 3. Evidence-based indicators for maturity assessment of smart seismic technologies.

Code	Indicator	Evidence source	Description
G1	Research visibility	Scientific evidence	General visibility of the technology in scholarly records
G2	Recent research growth	Scientific evidence	Recent increase in research attention, especially after 2020
G3	Experimental/validation evidence	Scientific evidence	Availability of laboratory, numerical, experimental, or validation-based evidence
G4	Implementation/case-study evidence	Scientific evidence	Evidence of real-world application, prototype use, field implementation, or case studies
G5	Recovery/resilience research evidence	Scientific evidence	Relevance to reparability, downtime reduction, re-occupancy, functional recovery, or seismic resilience
P1	Patent visibility	Patent evidence	General visibility of the technology in patent records
P2	Seismic/civil patent relevance	Patent evidence	Relevance of patent records to seismic, structural, or civil engineering applications
P3	Device/system maturity	Patent evidence	Evidence of device-level, system-level, or application-oriented technological development
P4	Recent patent signal	Patent evidence	Recent patent activity indicating continued technological development
P5	Recovery/resilience patent signal	Patent evidence	Patent evidence related to repair, rapid assessment, recovery, re-occupancy, or resilience functions

The indicators were then rated on an ordinal scale ranging from 1 to 6, where higher numbers indicated more maturity evidence. These scores do not represent a straightforward tally of papers or patents; instead, they represent maturity evidence in ordinal form that was derived from the screening process. Interpretation of this scale is outlined in Table 4.

Table 4. Interpretation of coding scores.

Score	Label	Interpretation
1	Low	Very limited evidence or weak relevance to smart seismic applications
2	Low-medium	Some evidence exists, but relevance, maturity, or application evidence is limited
3	Medium	Moderate evidence with partial relevance to seismic or smart infrastructure applications
4	Medium-high	Clear evidence of relevance, validation, or development activity
5	High	Strong evidence of research, validation, patent relevance, or application potential
6	Very high	Very strong evidence, high visibility, strong validation, or direct relevance to maturity/resilience

After the evidence matrix was coded, the CRITIC method was applied to determine the objective weight of each maturity indicator. CRITIC was selected because it considers both the contrast intensity of each indicator and the conflict among indicators. In this study, all indicators were treated as benefit-type indicators, meaning that higher coded values represented stronger maturity-related evidence.

First, the coded decision matrix was normalized using benefit-type normalization, as shown in Eq. 1.

$$x'_{ij} = \frac{x_{ij} - \min(x_j)}{\max(x_j) - \min(x_j)} \quad (1)$$

where x_{ij} is the coded value of technology i under indicator j , and x'_{ij} is the normalized value.

Second, the information content of each indicator was calculated as Eq. 2.

$$C_j = \sigma_j \sum_{k=1}^m (1 - r_{jk}) \quad (2)$$

where σ_j is the standard deviation of indicator j , r_{jk} is the correlation coefficient between indicators j and k , and m is the number of indicators.

Finally, the objective weight of each indicator was obtained as Eq. 3.

$$w_j = \frac{C_j}{\sum_{j=1}^m C_j} \quad (3)$$

where w_j is the CRITIC weight of indicator j . The weighted maturity score of each technology was then calculated by combining the normalized indicator values with the CRITIC weights, as shown in Eq. 4.

$$M_i = \sum_{j=1}^m w_j x'_{ij} \quad (4)$$

where M_i is the weighted maturity score of technology i .

The weighted CRITIC score was used to describe the overall evidence intensity of each technology group. However, the final maturity/application stage was not assigned solely based on a single threshold value of the total CRITIC score. This is because smart seismic technologies may show different maturity patterns. For example, a technology may have strong recent research growth but limited implementation evidence, while another technology may have stronger recovery/resilience evidence even if its overall score is not the highest. Therefore, the final classification was based on a profile-based interpretation of the CRITIC results. In this approach, the weighted score was considered together with the dominant evidence pattern across scientific growth, validation evidence, implementation evidence, patent relevance, device/system maturity, and recovery/resilience signals.

The classification rule was defined as follows. Technologies with strong recent scientific growth but limited implementation and device-level evidence were classified as M1 emerging/research-oriented. Technologies with stronger validation, monitoring, prototype, patent, or technology-readiness signals were classified as M2 developing/technology-readiness. Technologies with stronger implementation, design, deployment, and engineering application evidence were classified as M3 implementation-oriented. Technologies whose dominant evidence profile was related to reparability, re-occupancy, downtime reduction, rapid damage assessment, functional recovery, or resilience were classified as M4 recovery/resilience-oriented. Based on the weighted evidence profiles, the technologies were assigned to four maturity/application stages: emerging/research-oriented, developing/technology-readiness, implementation-oriented, and recovery/resilience-oriented. These stages were not treated as formal Technology Readiness Levels. Instead, they were used as planning-oriented maturity categories designed to support foresight portfolio selection.

In this framework, emerging/research-oriented technologies are those with strong scientific potential and recent research growth but limited large-scale implementation evidence. Developing/technology-readiness technologies show stronger evidence of validation, prototype development, patent activity, monitoring applications, or increasing readiness signals. Implementation-oriented technologies are closer to engineering deployment and are associated with feasibility, stakeholder acceptance, practical

application, and design or retrofit relevance. Recovery/resilience-oriented technologies are mainly linked to repairability, downtime reduction, re-occupancy, rapid damage assessment, and post-earthquake functional recovery.

This evidence-based maturity structure provides the foundation for linking each smart seismic technology stage with suitable foresight methods. Therefore, the CRITIC-based maturity assessment does not only classify technologies, but also supports the main logic of the study: foresight methods should be selected and combined according to the maturity and application context of smart seismic technologies.

4.2. Expert questionnaire survey and decision criteria

After defining the maturity/application stages of smart seismic technologies, an expert questionnaire survey was conducted to evaluate the suitability of foresight methods. The expert-based stage was designed to capture judgments on three aspects: the relative importance of decision criteria, the performance of foresight methods under these criteria, and the suitability of foresight-method portfolios for different technology maturity stages.

Twelve validated expert questionnaires have been used for the analysis. Each expert held a doctorate degree in civil engineering in the fields of structural engineering and earthquake engineering. This kind of expertise was relevant for the research area under investigation, particularly in relation to seismic performance, smart structural systems, structural health monitoring, seismic resilience, and engineering decision-making. For confidentiality purposes, no names or institutions are listed.

The questionnaire consisted of three main analytical parts. In the first part, the experts ranked the decision criteria and provided comparative judgments for SWARA-based criteria weighting. In the second part, they evaluated the performance of eight foresight methods with respect to the decision criteria using linguistic ratings. In the third part, they assessed suitable foresight-method portfolios for each maturity/application stage. This part allowed tied ranks, meaning that more than one foresight method could be assigned to the same importance level when experts considered them equally relevant.

Eight decision criteria have been set in order to evaluate foresight approaches for use in smart seismic technology planning (Table 5). These decision criteria have been selected with a view to meeting both the general requirements for foresight approaches and special requirements of smart seismic buildings.

Table 5. Decision criteria used for evaluating foresight methods.

Code	Criterion	Description
C1	Emerging trend identification	Ability to identify emerging scientific and technological trends
C2	Weak signal detection	Ability to detect weak signals and disruptive future changes
C3	Technology maturity and readiness assessment	Ability to assess technology maturity, readiness, and development potential
C4	Applicability to smart seismic technologies	Suitability for smart seismic systems and earthquake engineering applications
C5	Seismic resilience support	Ability to support damage reduction, repairability, downtime reduction, re-occupancy, and functional recovery
C6	Stakeholder acceptance and practical usability	Ability to consider stakeholder acceptance, usability, and implementation feasibility
C7	Long-term uncertainty handling	Ability to address long-term technological and contextual uncertainty
C8	Implementation planning and roadmapping support	Ability to support staged implementation, planning, and technology roadmapping

The foresight alternatives included eight methods commonly used in technology foresight, innovation planning, and strategic decision support (Table 6). These methods were selected because they represent complementary foresight functions, ranging from scientific trend analysis and patent-based innovation tracking to expert consensus, weak-signal detection, stakeholder analysis, scenario exploration, and implementation planning.

Table 6. Foresight methods evaluated in the expert questionnaire.

Code	Foresight method	Main role in the framework
A1	Brainstorming	Generating ideas and exploring early expert insights
A2	MACTOR/stakeholder analysis	Evaluating actor interests, conflicts, and implementation feasibility
A3	Bibliometrics	Identifying scientific trends and emerging research domains
A4	Patent analysis	Assessing innovation activity and technology development signals
A5	Delphi method	Building structured expert consensus under uncertainty
A6	Horizon scanning	Detecting weak signals and emerging changes
A7	Scenario planning	Exploring alternative future conditions and uncertainty
A8	Technology roadmapping	Translating foresight results into staged development and implementation pathways

For the performance evaluation of foresight methods, a five-level linguistic scale was used: very low (VL), low (L), medium (M), high (H), and very high (VH). These linguistic ratings were converted into fuzzy numbers in the Fuzzy VIKOR analysis. This allowed the study to model the vagueness of expert judgments while ranking foresight methods across multiple criteria.

The portfolio assessment part of the questionnaire was designed to provide assistance for making decisions taking into consideration the level of maturity. In contrast to forcing the experts to define a single approach at every level of maturity, the tied-

rank design made it possible to select combinations of approaches that have different weights. The above-mentioned approach is consistent with the main goal of the research, which is developing foresight portfolios in accordance with different levels of maturity of the technology under consideration.

4.3. SWARA-based criteria weighting

After defining the decision criteria, the SWARA method was applied to determine their relative weights based on expert judgments. SWARA was selected because it allows experts to express the importance of criteria in a stepwise and interpretable manner, making it suitable for decision problems where expert knowledge plays a central role [35]. SWARA's appropriateness for developing the framework for weighting expert judgments has been proven in many previous studies carried out by the researchers [41–44]. In this study, the method was used to weight the eight criteria $C_1 - C_8$, which represent the main requirements for selecting foresight methods for smart seismic technology planning.

According to the questionnaire, the criteria were ranked by experts according to their level of importance, starting with the most important. Further on, for every subsequent criterion, which was ranked starting with the second one, the expert stated the importance ratio coefficient s_j , which reflected the relative importance of criterion j compared to criterion $j - 1$.

For each expert, the SWARA procedure was implemented in three steps. First, the coefficient k_j was calculated as Eq. 5.

$$k_j = \begin{cases} 1, & j = 1 \\ s_j + 1, & j > 1 \end{cases} \quad (5)$$

where s_j is the comparative importance coefficient assigned by the expert. Second, the recalculated weight q_j was obtained as Eq. 6.

$$q_j = \begin{cases} 1, & j = 1 \\ \frac{q_{j-1}}{k_j}, & j > 1 \end{cases} \quad (6)$$

Finally, the normalized criterion weight w_j was calculated as Eq. 7.

$$w_j = \frac{q_j}{\sum_{j=1}^n q_j} \quad (7)$$

where n is the number of criteria. This procedure was applied separately to the responses of the twelve experts. The individual weights were then aggregated by averaging the corresponding criterion weights across experts and normalizing them to obtain the final SWARA-based criterion weights.

The resulting weights reflect the collective expert judgment regarding the relative importance of the criteria. These weights were subsequently used as input for the Fuzzy VIKOR analysis to rank the foresight methods. In this way, the SWARA-based weighting step connects expert judgment on decision priorities with the later evaluation of foresight alternatives.

4.4. Fuzzy VIKOR ranking of foresight methods

After obtaining the SWARA-based criterion weights, the foresight methods were ranked using the Fuzzy VIKOR method. VIKOR was selected because it identifies compromise solutions among alternatives evaluated under multiple and potentially conflicting criteria [36]. In this study, the alternatives were the eight foresight methods $A_1 - A_8$, and the criteria were the eight decision criteria $C_1 - C_8$.

Since expert assessments could be represented linguistically, the fuzzy scale was used due to the natural vagueness of such assessments. The linguistic terms have been converted into triangular fuzzy numbers as shown in Table 7.

Table 7. Linguistic scale used for Fuzzy VIKOR evaluation.

Linguistic term	Code	Triangular fuzzy number
Very low	VL	(0.00 , 0.00 , 0.25)
Low	L	(0.00 , 0.25 , 0.50)
Medium	M	(0.25 , 0.50 , 0.75)
High	H	(0.50 , 0.75 , 1.00)
Very high	VH	(0.75 , 1.00 , 1.00)

In every foresight approach, the linguistic evaluation results provided by the twelve experts were first transformed into triangular fuzzy numbers. After that, the aggregated fuzzy values were calculated for each alternative on each criterion based on the evaluations of the experts. Assume that a triangular fuzzy number is represented as $\tilde{x}_{ij} = (l_{ij}, m_{ij}, u_{ij})$. The aggregated fuzzy value was determined by Eq. 8.

$$x_{ij} = \left(\frac{1}{p} \sum_{e=1}^p l_{ij}^e, \frac{1}{p} \sum_{e=1}^p m_{ij}^e, \frac{1}{p} \sum_{e=1}^p u_{ij}^e \right) \quad (8)$$

where p is the number of experts, and l , m , and u represent the lower, middle, and upper values of the triangular fuzzy number.

To apply VIKOR, the aggregated fuzzy ratings were defuzzified into crisp scores. The centroid method was used (Eq. 9).

$$x_{ij} = \frac{l_{ij} + m_{ij} + u_{ij}}{3} \quad (9)$$

where x_{ij} is the defuzzified performance score of foresight method i under criterion j . Since all criteria in this study were benefit-type criteria, higher values indicated better performance.

Next, the best and worst values for each criterion were identified as shown in Eqs. 10 and 11.

$$f_j^* = \max_i x_{ij} \quad (10)$$

$$f_j^- = \min_i x_{ij} \quad (11)$$

where f_j^* and f_j^- represent the best and worst performance values under criterion j , respectively.

The VIKOR utility measure S_i and regret measure R_i were then calculated for each foresight method, using Eqs. 12 and 13.

$$S_i = \sum_{j=1}^n w_j \frac{f_j^* - x_{ij}}{f_j^* - f_j^-} \quad (12)$$

$$R_i = \max_j \left[w_j \frac{f_j^* - x_{ij}}{f_j^* - f_j^-} \right] \quad (13)$$

where w_j is the SWARA-based weight of criterion j . The S_i value represents the overall distance of an alternative from the ideal solution, while R_i represents the maximum individual regret across criteria.

Finally, the VIKOR compromise index Q_i was computed as Eq. 14.

$$Q_i = v \frac{S_i - S^*}{S^- - S^*} + (1 - v) \frac{R_i - R^*}{R^- - R^*} \quad (14)$$

where:

$$S^* = \min_i S_i, \quad S^- = \max_i S_i \quad (15)$$

The parameter v refers to the weighting given to the majority utility approach, while the parameter $1 - v$ indicates the weighting given to the individual regret approach. For the main study, $v = 0.5$ was used to balance the two attitudes towards decision making. The lower Q_i values indicate better performance in compromising. Therefore, the foresight techniques were sorted according to their Q_i values.

Fuzzy VIKOR methodology produced a ranking of the foresight methods with respect to all the criteria used in decision making. This ranking was not considered the final outcome of the study. It only acted as an interim outcome, which was combined later with the Borda ranking of ties to produce foresight portfolios suitable for different stages of smart earthquake technology development.

4.5. Tied-rank Borda portfolio extraction

After the overall ranking of foresight methods was obtained through Fuzzy VIKOR, a tied-rank Borda analysis was applied to identify suitable foresight portfolios for each maturity/application stage. This step was necessary because the objective of the study was not to recommend a single foresight method for all smart seismic technologies. Instead, the aim was to determine which combinations of foresight methods are more suitable for emerging, developing, implementation-oriented, and recovery/resilience-oriented technologies.

The questionnaire included a portfolio assessment part where the relevancy of eight foresight methods for each level of maturity was assessed by experts. Experts were allowed to assign tied ranks where several methods could be ranked the same in terms of their appropriateness because it was considered appropriate in this case, as foresight methods are usually complementary rather than competitive. The scoring scheme used in the tied-rank Borda analysis is shown in Table 8.

Table 8. Scoring scheme for tied-rank Borda portfolio extraction.

Expert-assigned level	Interpretation	Borda score
Rank 1	Essential method	3
Rank 2	Supporting method	2
Rank 3	Optional method	1
Not selected	Not included in the portfolio	0

For each maturity stage, the Borda score of foresight method i was calculated by aggregating the scores assigned by the twelve experts, as shown in Eq. 16.

$$B_{im} = \sum_{p=1}^e b_{ime} \quad (16)$$

where B_{im} is the total Borda score of foresight method i for maturity stage m , b_{ime} is the score assigned by expert e , and p is the number of experts. To support comparison across stages, the average Borda score was also calculated as Eq. 17.

$$\bar{B}_{im} = \frac{B_{im}}{p} \quad (17)$$

A higher Borda score indicates stronger expert support for including a method in the foresight portfolio of a given maturity stage. Methods with the highest scores were selected as the recommended portfolio for that stage.

The tied-rank Borda analysis complements the Fuzzy VIKOR ranking. While VIKOR provides an overall compromise ranking of foresight methods across all criteria, Borda analysis captures the stage-specific suitability of methods. This distinction is important because a method may not be highly ranked in the overall VIKOR result but may still be highly relevant for a specific maturity stage. For example, bibliometrics and patent analysis may be especially useful for emerging or developing technologies, even if expert consensus methods or roadmapping methods show stronger overall performance.

Therefore, the Borda step converts expert judgments into maturity-sensitive foresight portfolios. These portfolios provide the main practical output of the framework by linking each technology maturity stage with a suitable combination of foresight methods.

4.6. Expert agreement, reliability, and robustness analyses

To strengthen the credibility of the expert-based decision process, this study included both statistical checks of expert responses and robustness analyses of the ranking results. Since the proposed framework relies on judgments from twelve experts, it was necessary to examine whether the expert inputs showed an acceptable level of agreement and internal consistency before interpreting the final SWARA, Fuzzy VIKOR, and Borda results. Therefore, expert agreement, questionnaire reliability, uncertainty analysis, sensitivity analysis, and comparative validation were considered as complementary validation steps.

In the first step, Kendall's coefficient of concordance was used to assess the degree of agreement between the experts in their rankings. The use of Kendall's W is justified since it can be used to test the consistency of multiple experts' rankings of the same items. The measure was used to test the rankings of SWARA criteria and the maturity stage portfolio. Kendall's W scale ranges between 0 and 1, where a value closer to 1 shows a higher degree of agreement between experts. Statistical significance of Kendall's W was also tested.

Second, Cronbach's alpha was calculated as a supplementary reliability indicator for the linguistic evaluation matrix used in the Fuzzy VIKOR analysis. While Kendall's W evaluates agreement among experts in ranking-based tasks, Cronbach's alpha assesses the internal consistency of questionnaire responses. In this study, Cronbach's alpha was used to examine whether the expert ratings across the foresight-method evaluation matrix were sufficiently consistent for multi-criteria analysis.

Once the agreement of the experts was verified and the questionnaire validated, an uncertainty analysis using the Monte Carlo method was carried out to test the robustness of the results in light of the uncertainty associated with the expert opinion and linguistic evaluation. The simulation was run 10,000 times. During each of those times, the expert answers were resampled, and the linguistic evaluations were randomly altered within their corresponding fuzzy intervals. After that, the criterion weights based on SWARA and the ranking indicators based on VIKOR were calculated again. With the help of this procedure, the probabilities of each foresight technique remaining in the lead (for example, having ranks 1, 2, or 3) could be estimated.

Sensitivity analysis was then performed on the VIKOR compromise coefficient. In the main analysis, the coefficient was set to 0.50 to balance the majority utility strategy and the individual regret strategy. To test whether the ranking structure was sensitive to different decision attitudes, the analysis was repeated using values of 0.25, 0.50, and 0.75. This step helped determine whether the priority order of foresight methods changed when greater emphasis was placed on either group utility or individual regret.

Finally, the comparative validation was done using TOPSIS and PROMETHEE II. Both techniques have been used because each one represents a different logic of multi-criteria decision making. TOPSIS uses the proximity of the alternatives to the ideal and anti-ideal solutions to rank them, while PROMETHEE II provides a comprehensive ranking on the basis of preference flow [36, 38]. The rankings from both these techniques have been compared to the ranking provided by Fuzzy VIKOR for evaluating that if the results depend on any one decision-making method.

Through these checks, the study evaluates both the reliability of expert-based inputs and the robustness of the final decision results. Kendall's W and Cronbach's alpha support the credibility of the questionnaire-based judgments, while Monte Carlo simulation, VIKOR sensitivity analysis, TOPSIS, PROMETHEE II, and Spearman correlation assess the stability and external consistency of the ranking and portfolio outputs. Therefore, this validation structure strengthens the reliability of the proposed maturity-sensitive foresight portfolio framework.

5. Results and discussion

This section presents and discusses the results of the proposed maturity-based foresight portfolio framework. The results are organized according to the methodological workflow of the study, moving from evidence-based technology maturity assessment to expert-based foresight evaluation, portfolio extraction, and robustness validation. First, the CRITIC results are reported to show how smart seismic technologies were classified into maturity/application stages. Second, the SWARA-based criterion weights are presented to identify the most influential factors in selecting foresight methods. Third, the Fuzzy VIKOR ranking is discussed to determine the overall priority of foresight methods across the decision criteria. Fourth, the tied-rank Borda results are used to extract maturity-sensitive foresight portfolios for each technology stage. Finally, Monte Carlo uncertainty analysis, VIKOR sensitivity analysis, and comparative validation with TOPSIS and PROMETHEE II are discussed to evaluate the robustness of the findings.

The purpose of this section is not only to report numerical results, but also to interpret their implications for smart seismic

technology planning. In particular, the discussion focuses on how different maturity stages create different foresight needs. Emerging technologies require methods that can identify scientific trends and weak signals; developing technologies require tools for assessing readiness and innovation potential; implementation-oriented technologies require stakeholder and deployment analysis; and recovery-oriented technologies require methods that can address reparability, downtime reduction, re-occupancy, and functional recovery. Therefore, the results are interpreted with respect to the main objective of the study: developing maturity-sensitive foresight portfolios rather than selecting a single universal foresight method.

5.1. CRITIC-based maturity assessment results

The CRITIC-based maturity assessment was conducted using the coded ordinal evidence matrix developed from the structured screening of 200 scientific records and 100 patent records. Therefore, the maturity results should be interpreted as evidence-based ordinal maturity profiles rather than as direct publication counts or patent counts. The purpose of this analysis was to identify the relative maturity/application profile of smart seismic technology groups and to classify them into planning-oriented maturity stages for foresight portfolio selection.

The CRITIC method was applied to ten maturity-related indicators, including five scientific indicators and five patent-based indicators. The scientific indicators captured research visibility, recent research growth, experimental or validation evidence, implementation or case-study evidence, and recovery/resilience research evidence. The patent-based indicators captured patent visibility, seismic/civil patent relevance, device or system maturity, recent patent activity, and recovery/resilience patent signals. Since CRITIC assigns higher weights to indicators with greater contrast and lower redundancy, the resulting weights indicate which evidence dimensions provided the most useful information for distinguishing the maturity profiles of smart seismic technologies (Table 9).

Table 9. CRITIC weights of evidence-based maturity indicators.

Rank	Code	Indicator	Evidence source	CRITIC weight
1	G2	Recent research growth	Scientific evidence	0.234
2	P5	Recovery/resilience patent signal	Patent evidence	0.130
3	G5	Recovery/resilience research evidence	Scientific evidence	0.116
4	G1	Research visibility	Scientific evidence	0.093
5	G3	Experimental/validation evidence	Scientific evidence	0.085
6	P2	Seismic/civil patent relevance	Patent evidence	0.075
7	P4	Recent patent signal	Patent evidence	0.072
8	P1	Patent visibility	Patent evidence	0.070
9	P3	Device/system maturity	Patent evidence	0.064
10	G4	Implementation/case-study evidence	Scientific evidence	0.062

The highest weight was assigned to recent research growth ($G2 = 0.234$). This indicates that recent growth in scientific attention was the most informative indicator for distinguishing the maturity profiles of smart seismic technologies. This result is reasonable because several smart seismic technologies, such as AI-based damage detection, digital twins, advanced sensing systems, and IoT-based seismic monitoring, are characterized by rapid research development. Strong recent research growth does not necessarily mean that a technology is fully mature or widely implemented, but it is an important signal for identifying emerging and developing technology areas that require foresight attention.

The second and third highest weights were assigned to recovery/resilience patent signal ($P5 = 0.130$) and recovery/resilience research evidence ($G5 = 0.116$). These results show that resilience-oriented evidence played an important role in the maturity assessment. In the context of smart seismic structures, maturity should not be evaluated only through general research visibility or technical development. It should also consider whether a technology supports reparability, downtime reduction, rapid damage assessment, re-occupancy, functional recovery, and post-earthquake resilience. Therefore, the relatively high weights of $P5$ and $G5$ confirm that recovery and resilience functions are central to the maturity classification of smart seismic technologies.

Research visibility ($G1 = 0.093$) and experimental/validation evidence ($G3 = 0.085$) also received meaningful weights. These indicators helped distinguish technologies with broad scholarly attention and stronger validation evidence from technologies that remain more conceptual or early-stage. Patent-related indicators also contributed to the applied and innovation-oriented side of the maturity assessment. Seismic/civil patent relevance, recent patent signal, patent visibility, and device/system maturity helped identify whether a technology showed evidence of practical embodiment, device-level development, system-level design, or application-oriented innovation.

The lowest CRITIC weight was assigned to implementation/case-study evidence ($G4 = 0.062$). This does not mean that implementation evidence is unimportant. Rather, it indicates that, within the coded evidence matrix, this indicator provided less independent contrast than indicators such as recent research growth and recovery/resilience signals. In practical interpretation, implementation evidence remains important for identifying technologies that are closer to engineering deployment, but it should be considered together with validation evidence, patent relevance, device maturity, and resilience-related signals.

Although weighted CRITIC scores were calculated for all technology groups, the final maturity/application classification was

not based only on a fixed numerical threshold. The classification was profile-based. In other words, the weighted score was used to describe the overall evidence intensity of each technology group, while the final stage assignment was determined by the dominant evidence pattern across research growth, validation evidence, implementation evidence, patent relevance, device/system maturity, and recovery/resilience signals. This distinction is important because smart seismic technologies may show different maturity patterns. For example, digital twins may show strong recent research growth and future potential while still having limited routine implementation in seismic engineering practice. Conversely, recovery-oriented tools may be classified as recovery/resilience-oriented because their dominant evidence profile is linked to repairability, re-occupancy, downtime reduction, and functional recovery.

Table 10 clarifies that the maturity/application classification was not threshold-based. A higher weighted CRITIC score indicates stronger overall evidence intensity, but it does not automatically place a technology in a later maturity stage. Instead, each technology was assigned to a stage according to its dominant evidence profile. This profile-based interpretation is suitable for smart seismic technologies because some technologies are mainly research-intensive, some are readiness-oriented, some are closer to engineering implementation, and others are primarily linked to post-earthquake recovery and resilience objectives.

Table 10. Profile-based classification of smart seismic technologies using CRITIC results.

Technology group	Weighted CRITIC score	Dominant evidence profile	Classification rule	Final stage
AI-based damage detection	0.328	High recent research growth; limited implementation and device maturity	Strong scientific emergence but limited deployment evidence	M1. Emerging/research-oriented
Digital twins	0.401	High recent research growth and recent patent signal; limited field implementation in seismic practice	Strong research and future potential, but limited routine seismic implementation	M1. Emerging/research-oriented
SHM/advanced smart sensing	0.504	Strong research visibility, validation evidence, patent relevance, and monitoring applications	Stronger validation and technology-readiness signals	M2. Developing/technology-readiness
Smart sensors/IoT-based seismic monitoring	0.599	Strong recent research growth, validation, monitoring applications, and patent activity	Strong technology-readiness and monitoring-application profile	M2. Developing/technology-readiness
MR dampers/semi-active control	0.270	Strong validation, patent relevance, and device/system maturity	Device-level and control-system readiness, but requiring broader application planning	M2. Developing/technology-readiness
Self-centering systems	0.331	Strong validation and implementation relevance; direct connection to seismic design and residual drift control	Engineering application and deployment-oriented evidence	M3. Implementation-oriented
Base isolation systems	0.446	Very strong validation and implementation/case-study evidence	Established engineering application and deployment relevance	M3. Implementation-oriented
Supplemental damping devices	0.561	Strong validation, patent relevance, device maturity, and implementation evidence	Strong implementation-oriented seismic protection profile	M3. Implementation-oriented
Replaceable fuse/low-damage systems	0.395	Very strong recovery/resilience research evidence and strong recovery patent signal	Dominant repairability and low-damage recovery function	M4. Recovery/resilience-oriented
Rapid damage assessment/re-occupancy tools	0.527	Very strong recovery/resilience research and patent signals	Dominant post-earthquake assessment, re-occupancy, and functional recovery profile	M4. Recovery/resilience-oriented

Based on this profile-based interpretation, the selected smart seismic technologies were classified into four maturity/application stages (Table 11). These stages were not treated as formal technology readiness levels, but as planning-oriented categories designed to support maturity-sensitive foresight portfolio selection.

Table 11. CRITIC-based maturity/application classification of smart seismic technologies.

Maturity stage	Technology examples	Main maturity interpretation
M1. emerging/research-oriented	AI-based damage detection; Digital twins	Strong research potential and rapid scientific growth, but limited large-scale implementation evidence
M2. developing/technology-readiness	Advanced smart sensing; SHM; Smart sensors/IoT-based seismic monitoring; MR dampers/semi-active control	Increasing validation, prototype development, monitoring applications, and technology-readiness signals
M3. implementation-oriented	Self-centering systems; Base isolation systems; Supplemental damping devices	Closer to engineering application, supported by stronger performance evidence and deployment relevance
M4. recovery/resilience-oriented	Replaceable fuse/low-damage systems; Rapid post-earthquake damage assessment; Re-occupancy/downtime reduction systems	Directly related to repairability, downtime reduction, re-occupancy, and functional recovery

The classification shows that smart seismic technologies do not follow a single maturity pathway. AI-based damage detection and digital twins were classified as emerging/research-oriented technologies because they show strong scientific potential and rapid research growth, but still require broader validation, standardization, and integration into routine seismic engineering practice. These technologies are important for future smart infrastructure systems, but their widespread implementation in seismic applications remains developing.

Structural health monitoring, advanced smart sensing, IoT-based seismic monitoring, and MR dampers/semi-active control were classified as developing/technology-readiness technologies. These technologies have stronger validation evidence, monitoring applications, prototype development, and patent-related signals than purely emerging technologies. However, they still require further integration into design practice, infrastructure management systems, and long-term implementation frameworks.

Self-centering systems, base isolation systems, and supplemental damping devices were classified as implementation-oriented technologies. These technologies are more directly connected to seismic response reduction, performance-based design, and engineering deployment. Their maturity profiles are supported by stronger experimental, analytical, and application-oriented evidence. However, their future adoption still depends on factors such as cost, constructability, maintenance requirements, code acceptance, and stakeholder confidence.

Replaceable fuse and low-damage systems, rapid post-earthquake damage assessment tools, and re-occupancy or downtime-reduction systems were classified as recovery/resilience-oriented technologies. These technologies are directly linked to post-earthquake reparability, downtime reduction, re-occupancy decisions, and functional recovery. Their maturity profile is therefore not defined only by technical readiness, but also by their contribution to resilience-oriented performance objectives.

Overall, the CRITIC results provide the evidence-based foundation for the subsequent expert-based foresight analysis. The maturity classification shows that different smart seismic technologies require different foresight needs. Emerging technologies require methods that can identify scientific trends and weak signals. Developing technologies require tools for assessing readiness, patent evidence, and transition pathways. Implementation-oriented technologies require stakeholder and deployment analysis. Recovery/resilience-oriented technologies require methods that can address uncertain post-earthquake conditions, reparability, downtime reduction, re-occupancy, and functional recovery. Therefore, the CRITIC-based maturity assessment supports the main logic of this study: foresight methods should be selected and combined according to the maturity and application context of smart seismic technologies.

5.2. SWARA-based criteria weights

After defining the maturity stages of smart seismic technologies, the SWARA-based weighting procedure was applied to determine the relative importance of the eight decision criteria (Table 12). The results reflect the collective judgment of the twelve experts regarding which criteria are most important for selecting foresight methods in the context of smart seismic technology planning.

Table 12. Final SWARA-based weights of decision criteria.

Rank	Code	Criterion	Final weight
1	C5	Seismic resilience support	0.181
2	C8	Implementation planning and roadmapping support	0.151
3	C4	Applicability to smart seismic technologies	0.145
4	C3	Technology maturity and readiness assessment	0.141
5	C6	Stakeholder acceptance and practical usability	0.117
6	C7	Long-term uncertainty handling	0.095
7	C1	Emerging trend identification	0.088
8	C2	Weak signal detection	0.082

The highest weight was assigned to seismic resilience support ($C5 = 0.181$). This result indicates that experts considered the ability of a foresight method to support resilience-oriented objectives as the most important selection criterion. In the context of smart seismic structures, this means that foresight methods should not only identify promising technologies, but also help evaluate how these technologies contribute to damage reduction, reparability, downtime reduction, re-occupancy, and functional recovery. This finding is consistent with the broader shift in earthquake engineering from life-safety-based performance toward resilience-oriented planning.

The second most important criterion was implementation planning and roadmapping support ($C8 = 0.151$). This shows that experts valued foresight methods that can translate technological possibilities into staged development and deployment pathways. For smart seismic technologies, this is particularly important because many solutions require long-term coordination among researchers, engineers, infrastructure owners, technology developers, and regulatory bodies. A foresight method that only identifies future opportunities is therefore less useful unless it can also support practical planning and implementation.

The third and fourth criteria were applicability to smart seismic technologies ($C4 = 0.145$) and technology maturity and readiness assessment ($C3 = 0.141$). Their high weights show that foresight-method selection should be closely connected to the engineering characteristics and maturity status of the technologies being assessed. In other words, a suitable foresight method must be capable of addressing the specific requirements of smart seismic systems, such as sensing, monitoring, control, post-earthquake assessment, and recovery support. At the same time, it should help distinguish between technologies that are still research-oriented and those that are closer to engineering implementation.

Stakeholder acceptance and practical usability ($C6 = 0.117$) also received a relatively important weight. This result highlights that the adoption of smart seismic technologies depends not only on technical performance, but also on institutional acceptance, usability, cost, maintenance requirements, and compatibility with real engineering practice. This is especially relevant for implementation-oriented technologies, where deployment feasibility and stakeholder confidence may strongly influence future adoption.

The criteria related to long-term uncertainty handling ($C7 = 0.095$), emerging trend identification ($C1 = 0.088$), and weak signal

detection ($C2 = 0.082$) received lower but still meaningful weights. These criteria are particularly relevant when it comes to nascent technologies where scientific progress takes place quickly and where future paths are unpredictable. The relatively low weight that is given to these criteria does not mean that identifying trends and picking up weak signals is less important but simply that experts focus more on their role in building resilience and implementation.

Overall, the SWARA results show that foresight-method selection for smart seismic technologies should be guided primarily by resilience relevance, implementation support, engineering applicability, and technology maturity assessment. This weighting pattern supports the central logic of the study: foresight portfolios should be selected not only to identify future technologies, but also to support their practical development and contribution to resilient seismic infrastructure.

5.3. Fuzzy VIKOR ranking of foresight methods

After obtaining the SWARA-based criterion weights, the eight foresight methods were evaluated using the Fuzzy VIKOR method (Table 13). The main purpose of this step was to identify the overall compromise priority of foresight methods across all decision criteria. The VIKOR compromise coefficient was set to $v = 0.50$, giving equal importance to the group utility measure S and the individual regret measure R [36].

Table 13. Fuzzy VIKOR ranking of foresight methods.

Rank	Code	Foresight method	S	R	Q
1	A5	Delphi method	0.265	0.062	0.047
2	A8	Technology roadmapping	0.210	0.082	0.083
3	A7	Scenario planning	0.407	0.145	0.517
4	A6	Horizon scanning	0.530	0.145	0.621
5	A2	MACTOR/stakeholder analysis	0.590	0.145	0.672
6	A1	Brainstorming	0.736	0.145	0.796
7	A4	Patent analysis	0.673	0.167	0.836
8	A3	Bibliometrics	0.798	0.181	1.000

The results reveal that the Delphi method (A5) reaches the optimal compromise position in relation to all criteria, which is proven by the smallest Q value. It proves that the Delphi technique works effectively on all selected criteria mainly because of the ability to generate a consistent solution amidst uncertainty. This feature becomes especially significant in the field of intelligent seismic technology as it is dominated by uncertainty. This research finding shows how effective Delphi is in decision-making based on the criteria that have been applied using weights. This is because Delphi is very effective in creating consensus among experts in cases of uncertainty. This is important since, in the area of smart seismic technologies, many technologies operate on incomplete data and uncertain development.

Technology roadmapping (A8) ranked second and showed the lowest S value among all methods. This suggests that roadmapping performs strongly from the perspective of overall group utility. Its high ranking is consistent with the importance assigned to implementation planning, technology readiness, and practical deployment in the SWARA results. For smart seismic technologies, roadmapping is particularly valuable because it can translate future-oriented insights into staged development pathways, milestones, and implementation plans. The difference between Delphi and technology roadmapping approaches is small compared to the gap between the latter two. Thus, the result can be viewed as not suggesting that the Delphi approach is sufficient on its own. Instead, a combination of both the Delphi and technology roadmapping approaches is considered to form a solid core of foresight planning. The Delphi approach is more efficient in reaching expert agreement and reducing uncertainties, while the technology roadmapping approach allows for easier implementation of the decisions made.

Scenario planning (A7) was ranked third in the list. This is because scenario planning is very vital, especially when it comes to analyzing different future situations. It is important, especially in technology that involves recovery after earthquakes, reducing downtime, and re-occupancy. While scenario planning was not ranked above Delphi and roadmapping in the list, it is still an essential factor when there is a high level of uncertainty about future hazards.

Methods such as horizon scanning (A6) and MACTOR/stakeholder analysis (A2) obtained middle positions in the overall ranking. Horizon scanning is useful for detecting weak signals and emerging changes, while MACTOR is more relevant for analyzing stakeholder interests and implementation feasibility. Their middle ranking suggests that these methods may be more stage-specific than universally dominant. In other words, they may not be the strongest methods across all criteria, but they can be highly relevant for particular maturity stages. Patent analysis (A4) and bibliometrics (A3) received lower overall VIKOR rankings. This does not mean that they are unimportant. Their lower position reflects the fact that the overall expert-weighted criteria gave stronger priority to resilience support, implementation planning, and practical usability. However, bibliometrics and patent analysis remain essential for specific maturity contexts. Bibliometrics is particularly useful for emerging technologies, where scientific trend identification is needed, while Patent Analysis is important for developing technologies, where innovation activity and technology-readiness signals must be assessed.

Overall, the Fuzzy VIKOR results indicate that Delphi, technology roadmapping, and scenario planning have the strongest overall compromise performance. However, the ranking also confirms that no single foresight method can address all maturity

stages of smart seismic technologies. Therefore, the VIKOR results were used as an intermediate prioritization step, while the final maturity-sensitive foresight portfolios were extracted through the tied-rank Borda analysis in the next section.

5.4. Maturity-sensitive portfolio results using tied-rank Borda analysis

The Fuzzy VIKOR results provided the overall ranking of foresight methods, but the main objective of this study was to identify suitable foresight portfolios for different maturity stages of smart seismic technologies. For this reason, tied-rank Borda analysis was applied to the maturity-stage portfolio assessments provided by the experts. This step allowed more than one method to be assigned to the same importance level, reflecting the fact that foresight methods are complementary rather than mutually exclusive (Table 14).

Table 14. Recommended maturity-sensitive foresight portfolios.

Maturity stage	Technology examples	Recommended foresight portfolio
M1. Emerging/research-oriented	AI-based damage detection; Digital twins	Bibliometrics + horizon scanning + Delphi
M2. Developing/technology-readiness	Advanced smart sensing; SHM; Smart sensors/IoT-based seismic monitoring; MR dampers/semi-active control	Patent analysis + Delphi + technology roadmapping
M3. Implementation-oriented	Self-centering systems; Base isolation systems; Supplemental damping devices	MACTOR + technology roadmapping + Delphi
M4. Recovery/resilience-oriented	Replaceable fuse/low-damage systems; Rapid post-earthquake damage assessment; Re-occupancy/downtime reduction systems	Technology roadmapping + MACTOR + Delphi + scenario planning

The suggested portfolio for M1, which is represented by emergence and research types of technology, includes bibliometrics, horizon scanning, and Delphi. Such a combination is suitable for use because, as mentioned above, technologies of emergence require methods that can help to spot scientific trends, identify weak signals, and verify future directions through the opinion of experts. Bibliometrics enables scientific mapping, while horizon scanning enables the identification of signals of changes and disruptions. For M2 developing/technology-readiness technologies, the recommended portfolio includes patent analysis, Delphi, and technology roadmapping. This stage requires a stronger focus on innovation activity, technology readiness, and transition from research to application. Patent analysis helps identify invention-oriented and commercialization signals, Delphi supports expert interpretation of uncertain readiness evidence, and technology roadmapping translates these findings into development milestones and application pathways. For M3 implementation-oriented technologies, the recommended portfolio consists of MACTOR, technology roadmapping, and Delphi. Technologies in this stage are closer to engineering deployment; therefore, the main challenge is no longer only identifying future opportunities, but also understanding stakeholder interests, implementation constraints, and practical feasibility. MACTOR is useful for analyzing actors and institutional conditions, roadmapping supports staged deployment, and Delphi helps refine expert-based priorities.

For M4 recovery/resilience-oriented technologies, the recommended portfolio includes technology roadmapping, MACTOR, Delphi, and scenario planning. This is the most complex portfolio because recovery-oriented technologies must address post-earthquake uncertainty, repairability, downtime reduction, re-occupancy, and functional recovery. Scenario Planning is particularly important at this stage because post-earthquake recovery depends on uncertain future conditions, including hazard intensity, damage distribution, resource availability, and recovery expectations. These results show that the most suitable foresight methods change across technology maturity stages. Bibliometrics and patent analysis are not the highest-ranked methods in the overall VIKOR results, but they become essential in specific maturity contexts. This confirms that relying only on a single overall ranking could lead to incomplete recommendations. The tied-rank Borda results therefore provide the main practical contribution of the study by translating expert judgments into stage-specific foresight portfolios for smart seismic technology planning.

5.5. Robustness and validation results

To examine the credibility and stability of the results, this section reports four complementary checks: expert agreement and questionnaire reliability, Monte Carlo uncertainty analysis, VIKOR sensitivity analysis, and comparative validation using TOPSIS and PROMETHEE II. The first check evaluates whether the expert-based inputs were sufficiently consistent for multi-criteria analysis. The remaining checks examine whether the ranking of foresight methods and the resulting maturity-sensitive portfolios remain stable under input uncertainty, changes in the VIKOR compromise coefficient, and alternative decision-making models.

Together, these analyses provide a broader validation structure for the proposed framework. Rather than relying only on the main Fuzzy VIKOR and Borda outputs, the study evaluates the reliability of the expert judgments, the robustness of the rankings, and the consistency of the results across different analytical assumptions. This helps determine whether the final foresight portfolios can be considered dependable for maturity-sensitive smart seismic technology planning.

5.5.1. Expert agreement and questionnaire reliability

Before interpreting the robustness analyses, the reliability of the expert-based inputs was examined using Kendall's coefficient of concordance and Cronbach's alpha. This step was added to determine whether the judgments provided by the twelve experts showed an acceptable level of agreement and internal consistency. Kendall's W was used for ranking-based parts of the questionnaire, while Cronbach's alpha was applied to the linguistic evaluation matrix used in the Fuzzy VIKOR analysis.

From the results (Table 15), it can be seen that the reliability of the expert inputs is enough for the subsequent analysis. The

value of Kendall's W coefficient for the criteria prioritization from the SWARA approach was 0.862 and was statistically significant ($\chi^2 = 42.238$, $P < 0.001$). It shows that criteria prioritization was not random, and experts agreed. Thus, the reliability of criteria weightings used in the SWARA approach is justified.

The derived foresight-method rankings also showed a very strong level of agreement among experts, with Kendall's W equal to 0.949 ($P < 0.001$). This suggests that the experts had highly consistent views regarding the relative performance of the foresight methods across the decision criteria. In addition, Cronbach's alpha for the 64-item linguistic evaluation matrix was 0.790, indicating acceptable internal consistency of the questionnaire responses used for the Fuzzy VIKOR analysis.

For the maturity-stage portfolio assessments, Kendall's W values were 0.864 for M1, 0.851 for M2, 0.889 for M3, and 0.787 for M4. These values indicate strong agreement across all maturity stages. Therefore, the stage-specific portfolio recommendations were not based on highly dispersed or inconsistent expert judgments. Overall, these statistical checks provide additional support for the reliability of the expert-based inputs used in the SWARA weighting, Fuzzy VIKOR ranking, and tied-rank Borda portfolio extraction.

Table 15. Expert agreement and questionnaire reliability results.

Reliability check	Data used	Result	Interpretation
Kendall's W	SWARA criteria rankings	0.862	Strong expert agreement
Chi-square	SWARA criteria rankings	42.238	Statistically significant
P-value	SWARA criteria rankings	< 0.001	Non-random agreement
Kendall's W	Derived foresight-method rankings	0.949	Very strong expert agreement
P-value	Derived foresight-method rankings	< 0.001	Statistically significant agreement
Cronbach's alpha	64-item linguistic evaluation matrix	0.790	Acceptable internal consistency
Kendall's W	M1 portfolio assessment	0.864	Strong expert agreement
Kendall's W	M2 portfolio assessment	0.851	Strong expert agreement
Kendall's W	M3 portfolio assessment	0.889	Strong expert agreement
Kendall's W	M4 portfolio assessment	0.787	Strong expert agreement

5.5.2. Monte Carlo uncertainty analysis

A simulation using Monte Carlo analysis involving 10,000 repetitions was conducted for evaluating the effect of uncertainties in experts' evaluations and linguistic assessment (Table 16). In each repetition, the response of the experts was randomly generated, and the linguistic assessment was varied according to the corresponding fuzzy set. The new values of the VIKOR criteria were calculated to find out the probability of dominance.

Table 16. Monte Carlo uncertainty analysis of foresight-method rankings.

Method	Mean Q	95% uncertainty interval	P (Rank 1)	P (Top 2)	P (Top 3)
A5. Delphi method	0.028	0.000-0.087	88.65%	99.99%	100%
A8. Technology roadmapping	0.102	0.001-0.182	11.34%	99.78%	100%
A7. Scenario planning	0.394	0.187-0.588	0.01%	0.23%	77.59%
A2. MACTOR/stakeholder analysis	0.546	0.360-0.736	0%	0%	13.58%
A6. Horizon scanning	0.553	0.387-0.707	0%	0%	8.73%
A1. Brainstorming	0.780	0.668-0.896	0%	0%	0%
A4. Patent analysis	0.800	0.573-0.955	0%	0%	0.10%
A3. Bibliometrics	0.982	0.882-1.000	0%	0%	0%

The Monte Carlo results confirm the robustness of the leading methods. Delphi had the highest probability of being ranked first, while technology roadmapping was almost always placed within the top two. Scenario planning was not usually ranked first, but it had a high probability of remaining within the top three. This indicates that Delphi and technology roadmapping form the most stable overall foresight core, while scenario planning plays an important supporting role under uncertainty.

The simulation also shows that the lower global ranking of bibliometrics and patent analysis is stable in the overall VIKOR results. However, this does not reduce their stage-specific relevance. As shown in the Borda portfolio results, bibliometrics remains essential for emerging technologies, and patent analysis remains essential for developing/technology-readiness stages.

5.5.3. VIKOR sensitivity analysis

Sensitivity analysis was performed by changing the VIKOR compromise coefficient v . The main analysis used $v = 0.50$, while two additional settings, $v = 0.25$ and $v = 0.75$, were tested to examine whether the ranking changed under different decision attitudes (Table 17).

Ranking was quite stable across the three values for v . The main difference was that Delphi and technology roadmapping interchanged places when $v = 0.75$. This result shows that Delphi produces better results in the case when individual regret is more

valuable, while technology roadmapping dominates in the case when the weight of majority utility increases. The stable ranking in third place of scenario planning serves to support its role as a reliable assisting technique.

Table 17. VIKOR sensitivity analysis under different compromise coefficients.

Method	Rank at $v = 0.25$	Rank at $v = 0.50$	Rank at $v = 0.75$
A5. Delphi method	1	1	2
A8. Technology roadmapping	2	2	1
A7. Scenario planning	3	3	3
A6. Horizon scanning	4	4	4
A2. MACTOR/stakeholder analysis	5	5	5
A1. Brainstorming	6	6	7
A4. Patent analysis	7	7	6
A3. Bibliometrics	8	8	8

5.5.4. Comparative validation with TOPSIS and PROMETHEE II

For the evaluation of dependence of the findings on the VIKOR approach, the ranking has been compared to the rankings based on the use of TOPSIS and PROMETHEE II approaches. These approaches are built on different logic: TOPSIS uses distances from ideal and anti-ideal solutions, while PROMETHEE II utilizes preference flows.

The comparative results show a consistent ranking structure across the three methods (Table 18). Delphi and technology roadmapping remained the two leading methods, although their order changed between VIKOR and the comparative methods. Scenario planning remained third in all methods, confirming its stable intermediate position. Bibliometrics remained last in the overall ranking, which again suggests that its role is not as a universal foresight method, but as a stage-specific method for emerging technologies.

Table 18. Comparative validation of foresight-method rankings.

Method	VIKOR $v = 0.50$	TOPSIS	PROMETHEE II
A8. Technology roadmapping	2	1	1
A5. Delphi method	1	2	2
A7. Scenario planning	3	3	3
A2. MACTOR/stakeholder analysis	5	4	5
A6. Horizon scanning	4	6	4
A4. Patent analysis	7	5	6
A1. Brainstorming	6	7	7
A3. Bibliometrics	8	8	8

Spearman's rank correlation was used to quantify ranking agreement (Table 19).

Table 19. Spearman rank correlation for robustness validation.

Comparison	Spearman correlation
VIKOR $v = 0.50$ vs. VIKOR $v = 0.25$	1.000
VIKOR $v = 0.50$ vs. VIKOR $v = 0.75$	0.952
VIKOR $v = 0.50$ vs. TOPSIS	0.857
VIKOR $v = 0.50$ vs. PROMETHEE II	0.952

The high correlation values indicate that the main ranking pattern is not strongly dependent on a single decision model. Therefore, the validation results strengthen confidence in the robustness of the proposed framework. Overall, the analyses confirm that the final maturity-sensitive foresight portfolios are supported not only by the main Fuzzy VIKOR and Borda results, but also by uncertainty simulation, sensitivity testing, and comparative multi-criteria validation.

5.6. Decision guideline and practical implications

Based on the CRITIC maturity classification, SWARA-based criterion weights, Fuzzy VIKOR ranking, and tied-rank Borda portfolio results, a practical decision guideline was developed for selecting foresight portfolios for smart seismic technologies. The guideline connects each maturity/application stage with its main decision need and the most suitable combination of foresight methods (Table 20).

The guideline shows that foresight planning for smart seismic technologies should be adjusted to the maturity and application context of each technology group. For emerging technologies, the main task is not immediate deployment, but understanding the direction of scientific development and identifying early signals of future change. Therefore, bibliometrics, horizon scanning, and

Delphi are most suitable because they combine research trend mapping, weak-signal detection, and expert interpretation.

For developing technologies, the key issue is the transition from research to application. At this stage, patent analysis becomes important because it can reveal innovation activity and technology-readiness signals. However, patent evidence alone is not sufficient. Delphi is needed to interpret uncertain evidence, while technology roadmapping helps translate readiness signals into development milestones and implementation pathways.

For implementation-oriented technologies, the main challenge is practical deployment. Technologies such as self-centering systems, base isolation, and supplemental damping devices are technically more mature, but their adoption depends on stakeholder acceptance, design practice, cost, maintenance, and regulatory conditions. Therefore, MACTOR, technology roadmapping, and Delphi form a suitable portfolio for analyzing actors, planning implementation, and refining priorities.

For recovery/resilience-oriented technologies, foresight must address post-earthquake uncertainty. Repairability, downtime reduction, re-occupancy, and functional recovery depend on uncertain damage scenarios, resource availability, inspection capacity, and recovery expectations. For this reason, scenario planning is added to the portfolio together with technology roadmapping, MACTOR, and Delphi. This combination supports both strategic planning and adaptive decision-making for post-earthquake recovery.

Overall, the decision guideline demonstrates the practical value of treating foresight-method selection as a portfolio design problem. A single method cannot adequately support all stages of smart seismic technology development. Instead, maturity-sensitive portfolios provide a more flexible and context-specific basis for research planning, technology development, implementation strategy, and resilience-oriented infrastructure decision-making.

Table 20. Decision guideline for selecting maturity-sensitive foresight portfolios for smart seismic technologies.

Maturity stage	CRITIC-based smart seismic technologies	Main decision need	Recommended foresight portfolio	Practical guideline
M1. Emerging/research-oriented	AI-based damage detection; Digital twins	Identifying scientific trends, weak signals, and future research directions	Bibliometrics + horizon scanning + Delphi	Avoid premature implementation decisions; First map scientific development and weak signals, then validate future directions through expert consensus
M2. Developing/technology-readiness	Advanced smart sensing; SHM; Smart sensors/IoT-based seismic monitoring; MR dampers/semi-active control	Assessing readiness, patent activity, development feasibility, and transition to application	Patent analysis + Delphi + technology roadmapping	Evaluate innovation and readiness signals, interpret them through expert judgment, and translate them into a staged development roadmap
M3. Implementation-oriented	Self-centering systems; Base isolation systems; Supplemental damping devices	Evaluating implementation feasibility, stakeholder acceptance, maintenance, and deployment pathways	MACTOR + technology roadmapping + Delphi	Analyze stakeholder roles and constraints, define implementation milestones, and refine deployment priorities through expert consensus
M4. Recovery/resilience-oriented	Replaceable fuse/low-damage systems; Rapid post-earthquake damage assessment; Re-occupancy/downtime reduction systems	Addressing repairability, downtime reduction, re-occupancy, and functional recovery	technology roadmapping + MACTOR + Delphi + scenario planning	Combine roadmap planning, stakeholder analysis, expert consensus, and alternative recovery scenarios to support post-earthquake functionality

6. Conclusions

This study proposed a technology-maturity-based foresight portfolio framework for smart seismic structures. The framework was developed to address the need for a structured decision-support approach that links the maturity level of smart seismic technologies with suitable combinations of foresight methods. Instead of treating foresight-method selection as the choice of a single best method, the study conceptualized foresight as a portfolio design problem in which different maturity stages require different combinations of evidence-based, expert-based, stakeholder-oriented, and planning-oriented methods.

The first stage of the analysis used scientific and patent-based evidence and the CRITIC method to weight maturity indicators and construct weighted maturity profiles. Based on predefined profile-based classification rules, the selected smart seismic technologies were then assigned to four maturity/application stages. The results showed that smart seismic technologies differ considerably in their maturity profiles. AI-based damage detection and digital twins were classified as emerging/research-oriented technologies, while SHM, smart sensors, IoT-based seismic monitoring, and semi-active control systems were positioned as developing/technology-readiness technologies. Self-centering systems, base isolation, and supplemental damping devices were classified as implementation-oriented technologies, whereas replaceable fuse systems, rapid damage assessment tools, and re-occupancy or downtime-reduction systems were classified as recovery/resilience-oriented technologies. This classification confirms that smart seismic technologies should not be planned through a single uniform foresight approach.

The SWARA-based weighting results showed that seismic resilience support was the most important criterion for selecting foresight methods. This indicates that foresight planning for smart seismic technologies should not focus only on technological novelty or research visibility, but should also consider repairability, downtime reduction, re-occupancy, and functional recovery. Implementation planning, applicability to smart seismic technologies, and technology maturity assessment were also among the most influential criteria, showing that practical deployment and engineering relevance are central to foresight-method selection in

this domain.

The Fuzzy VIKOR results indicated that Delphi and Technology Roadmapping form the strongest overall foresight core. Delphi is particularly useful for developing structured expert consensus under uncertainty, while Technology Roadmapping helps translate foresight results into staged development and implementation pathways. Scenario Planning also showed an important supporting role, especially where future recovery conditions and long-term uncertainty must be considered. However, the results also demonstrated that no single foresight method is sufficient for all technology maturity stages.

The tied-rank Borda analysis provided the main portfolio-based output of the study. For emerging/research-oriented technologies, Bibliometrics, Horizon Scanning, and Delphi were identified as the most suitable portfolio because they support scientific trend mapping, weak-signal detection, and expert interpretation. For developing/technology-readiness technologies, Patent Analysis, Delphi, and Technology Roadmapping were recommended because this stage requires innovation evidence, readiness assessment, and transition planning. For implementation-oriented technologies, MACTOR, Technology Roadmapping, and Delphi were more appropriate because stakeholder acceptance, feasibility, and deployment pathways become central. For recovery/resilience-oriented technologies, the recommended portfolio included Technology Roadmapping, MACTOR, Delphi, and Scenario Planning, reflecting the need to address post-earthquake uncertainty, reparability, downtime reduction, re-occupancy, and functional recovery.

The robustness and validation analyses strengthened the reliability of these findings. Expert agreement and questionnaire reliability checks showed that the expert-based inputs were sufficiently consistent for multi-criteria analysis. Monte Carlo uncertainty analysis confirmed that Delphi and Technology Roadmapping remained the most stable overall foresight methods, while VIKOR sensitivity analysis and comparative validation with TOPSIS and PROMETHEE II showed that the main ranking structure was not dependent on a single decision-making method. These results support the credibility of the proposed maturity-sensitive foresight portfolios.

The main contribution of this study is the integration of technology maturity assessment and foresight portfolio selection for smart seismic structures. The proposed framework advances existing foresight and resilience planning studies by connecting scientific and patent-based maturity evidence, expert-based multi-criteria evaluation, portfolio extraction, and robustness validation in a single decision-support process. In practical terms, the framework can help engineers, infrastructure owners, policymakers, and resilience planners select foresight methods according to the maturity and application context of smart seismic technologies. This is particularly useful for long-term planning of resilient infrastructure, where emerging technologies, implementation constraints, stakeholder interests, and post-earthquake recovery requirements must be considered together.

Overall, the findings show that maturity-sensitive foresight portfolios provide a more flexible and context-specific basis for smart seismic technology planning than a single-method ranking approach. By linking foresight methods to technology maturity stages, the proposed framework supports more informed decisions about research prioritization, technology development, implementation pathways, and resilience-oriented infrastructure planning.

Statements & declarations

Author contributions

Shahin Lale Arefi: Investigation, Formal analysis, Data curation, Methodology, Validation, Resources, Writing - Review & Editing.

Mahdi Bitarafan: Conceptualization, Investigation, Methodology, Formal analysis, Project administration, Resources, Writing - Original Draft.

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Data availability

The data presented in this study will be available on interested request from the corresponding author.

Declarations

The authors declare no conflict of interest.

Appendix A. Coded evidence matrix used for CRITIC analysis

Table A.1 shows the coded evidence matrix used in the maturity assessment based on the CRITIC methodology. These values represent ordinal maturity ratings that have been generated based on the screening of evidence from scientific and patent sources. They are not directly derived from counts of publications or patents. Instead, they are coded ratings of the following aspects: research visibility, concurrent growth of research, evidence of validation, evidence of implementation, patent visibility, seismic/civil patents relevance, maturity of devices/systems, current patent activity, and resilience indicators.

Table A.1. Coded evidence matrix used for CRITIC-based maturity assessment.

Technology group	G1	G2	G3	G4	G5	P1	P2	P3	P4	P5
AI-based damage detection	5	6	4	2	3	3	3	2	4	2
Digital twins	5	6	3	2	3	3	2	3	5	3
SHM/advanced smart sensing	6	5	5	4	4	5	5	4	4	3
Smart sensors/IoT-based seismic monitoring	5	6	4	4	4	5	4	4	5	3
MR dampers/semi-active control	5	4	5	3	3	5	5	5	4	2
Self-centering systems	5	4	5	4	4	4	4	4	4	4
Base isolation systems	6	4	6	6	4	5	5	6	4	2
Supplemental damping devices	6	4	6	5	4	6	6	5	5	3
Replaceable fuse/low-damage systems	4	4	5	3	6	4	4	5	4	5
Rapid damage assessment/re-occupancy tools	5	5	4	3	6	4	3	4	4	6

Note. G1 = research visibility; G2 = recent research growth; G3 = experimental/validation evidence; G4 = implementation/case-study evidence; G5 = recovery/resilience research evidence; P1 = patent visibility; P2 = seismic/civil patent relevance; P3 = device/system maturity; P4 = recent patent signal; P5 = recovery/resilience patent signal.

The coding scale used in this matrix was as follows: 1 = low, 2 = low-medium, 3 = medium, 4 = medium-high, 5 = high, and 6 = very high. Higher values indicate stronger maturity-related evidence for the corresponding indicator.

Appendix B. Search strings and screening depth used for evidence coding

The search strings and screening depth used for the structured evidence screening are reported in Table B.1. The purpose of this appendix is to improve the transparency of the CRITIC-based maturity assessment by showing how the scientific and patent-based evidence was identified for each technology group. The screening was conducted at the technology-group level. For each group, the first twenty most relevant scientific records and the first ten most relevant patent records were screened.

Table B.1. Search strings and screening depth used for evidence coding.

Technology group	Scientific search string	Patent search string	Scientific records screened	Patent records screened
AI-based damage detection	"AI-based damage detection" OR "machine learning damage detection" AND earthquake AND structure OR seismic	"earthquake damage detection" OR "seismic damage detection" AND artificial intelligence OR machine learning	20	10
Digital twins	"digital twin" AND civil infrastructure AND earthquake OR seismic	"digital twin" AND structure AND seismic OR earthquake	20	10
SHM/advanced smart sensing	"structural health monitoring" AND earthquake OR seismic AND smart sensing	"structural health monitoring" AND seismic OR earthquake AND sensor	20	10
Smart sensors/IoT-based seismic monitoring	"IoT-based seismic monitoring" OR "smart sensor" AND earthquake AND structure	"IoT" AND earthquake monitoring AND structural sensor OR seismic sensor	20	10
MR dampers/semi-active control	"magnetorheological damper" AND seismic OR "semi-active control" AND earthquake	"magnetorheological damper" AND seismic control OR earthquake control	20	10
Self-centering systems	"self-centering system" AND seismic OR earthquake-resistant structure	"self-centering" AND seismic structure OR earthquake-resistant structure	20	10
Base isolation systems	"base isolation" AND seismic AND building OR bridge	"base isolation" AND earthquake OR seismic isolation AND structure	20	10
Supplemental damping devices	"supplemental damping" AND seismic OR earthquake engineering	"damping device" AND seismic OR earthquake AND structure	20	10
Replaceable fuse/low-damage systems	"replaceable fuse" AND seismic OR "low-damage system" AND earthquake	"replaceable fuse" AND seismic OR earthquake-resistant structure	20	10
Rapid damage assessment/re-occupancy tools	"rapid damage assessment" AND earthquake OR "re-occupancy" AND seismic resilience	"post-earthquake damage assessment" OR re-occupancy AND structure OR building	20	10
Total	—	—	200	100

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Mechanical Response of Segmental Tunnels Subjected to Reverse Faulting: Physical and Numerical Modeling

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ABSTRACT

Underground tunnels crossing active reverse faults are highly vulnerable to permanent ground deformation, which may significantly affect their structural integrity and operational performance. This study investigates the behavior of segmental tunnels subjected to reverse-fault displacement using an integrated approach combining centrifuge testing and three-dimensional finite element modeling. The numerical model developed in ABAQUS was verified against centrifuge experiments, showing excellent agreement with the experimental observations, with prediction errors of only 2% for maximum ground settlement and 4% for maximum tunnel crown displacement. Following model validation, a comprehensive parametric study was conducted to evaluate the influence of reverse fault angle (60°, 75°, and 85°), soil stiffness, tunnel overburden (10-20 m), soil friction angle, and tunnel diameter (5-10 m) on tunnel performance. The results demonstrate that increasing the reverse fault angle substantially intensifies ground settlement, tunnel crown displacement, tunnel S-shaped deformation, and lining ring rotation while expanding the fault-affected zone. Increasing soil stiffness reduces ground settlement but promotes more complete upward propagation of fault rupture and increases localized tunnel deformation. Increasing tunnel overburden decreases both ground settlement and tunnel displacement by dissipating fault-induced deformation within the surrounding soil mass. In contrast, changes in the soil friction angle alone have only a negligible influence on tunnel behavior. Furthermore, increasing tunnel diameter has a limited effect on global settlement but reduces the plastic strain concentration above the tunnel and modifies the fault rupture path.

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1. Introduction

Natural disasters cause a lot of damage to human society annually, and earthquakes are of great importance, especially in most countries like Iran. According to an old belief, underground structures are by far the most secure against earthquakes. However, there are many reports of earthquake-induced damage to tunnels, mines, and underground spaces, so seismic events are considered one of the major risk factors in tunnelling today [1]. The use of tunnels in urban communities is currently expanding. Among the applications of tunnels are road, railway, subway, sewage collection, high-voltage power cables, and drinking water transmission tunnels, which are known as lifelines. Many major cities in the world are built on sediment deposits and fault zones. Many world-class standards have only recommended avoiding the construction of structures adjacent to active faults as an engineering solution [2]. There are numerous lifelines in urban and non-urban areas that may be severely damaged by major earthquakes, and their rupture can aggravate earthquake damage. A study of the history of damage caused by destructive earthquakes in the world shows

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that the share of damage to lifelines is considerable. For example, in the 1989 Loma Prieta earthquake, 90% of the damage was related to direct and indirect effects of damage to lifelines; in the 1994 Northridge earthquake, more than 80%; in the 1995 Japanese Kobe earthquake, 75%; and in the 1999 Turkish Izmit earthquake, over 85% of damage was related to direct and indirect effects of damage to lifelines. When a subway line that is considered an important lifeline is damaged by an earthquake, it can have significant indirect effects on human life as well as on a nation's comprehensive transportation system. Earthquake-prone countries like Iran are witnessing several large and small earthquakes each year and must take action practically and seriously to be safe in the face of these damages to structures such as tunnels, bridges, roads, and pipelines called lifelines [3]. These lines compulsorily cross a wide geographical area, so they face different environmental and sub-terrestrial factors due to earthquake forces. The design and analysis of these lines requires recognition of the behaviour of related parameters in terms of structural engineering, and a lack of familiarity with them can face the design and execution with irreparable damage and hazards, especially due to earthquakes. Therefore, it is not always possible to avoid tunnelling through an active fault. Tunnel passing through a fault is the most important part of tunnel design in the face of earth rupture. Faults can cause lots of damage to tunnels in case of high activity. The study of tunnel behaviour when passing through faults during earthquakes is an almost unknown topic. In this case, the tunnel must be capable of withstanding fault displacement and only cause trivial damage. Investigation of passing through faults in underground structures, especially tunnels excavated in the soil, is very complex. The experience of surface faulting in recent earthquakes has led to paying more attention to the serious damage to infrastructure due to permanent deformation of the earth (especially surface faulting). In the meantime, the effect of the fault on important lifelines is of great importance due to its large extent and, consequently, the possibility of interaction with any particular fault. This research will examine the behaviour of buried structures located at the fault site using numerical modelling. For this purpose, various parameters such as tunnel diameter, applied overburden, and fault angle are considered as variables, and the effect of each parameter will be investigated by considering each of the other parameters as constant.

The theoretical foundations for evaluating continuum soil media under large plastic shear deformations were established through limit analysis and finite element theorems by Bottero et al. [4]. Concurrently, physical modeling of fault rupture propagation through alluvial soil deposits gained momentum through the physical centrifuge experiments of Roth et al. [5]. Empirical observations of seismic damage and dynamic structural responses in underground openings were documented by Dowding and Rozen [6]. Further parameterizations regarding influence zone geometry, strain localization, and surface rupture patterns in alluvial deposits overlying active dip-slip faults were comprehensively detailed by Cole and Lade [7]. As urban infrastructure expanded into seismically active regions, physical centrifuge testing was adapted to evaluate buried structures. Burrige et al. [8] performed landmark centrifuge tests to evaluate the physical performance of tunnel linings intersecting fault zones. In structural geology, Twiss and Moores [9] established comprehensive geometric and kinematic classifications of fault slip mechanisms. Bray et al. [10] developed non-linear finite element procedures and empirical frameworks to model earthquake fault rupture propagation through cohesive soil embankments. Boundary element formulations for faulted rock masses and excavations adjacent to non-linear faults were advanced by Beer and Poulsen [11], Fotoohi and Mitri [12]. Reconnaissance studies following major earthquakes in the late 1990s and early 2000s highlighted severe structural vulnerabilities in transportation lifelines. Asakura and Sato [13] reported tunnel lining failures following the 1995 Kobe earthquake. Allen and Cluff [14] updated engineering recommendations for active faults in dam foundations. Post-earthquake field evaluations by Ulusay et al. [15] in Turkey, damage assessments by Kontogianni and Stiros [16], and physical scale modeling by Jeon et al. [17] confirmed that underground conduits experience massive stress concentrations, cross-sectional ovalization, and severe bending moments near active fault crossings. During the 2000s and 2010s, physical centrifuge modeling transitioned into highly instrumented 3D testing paired with numerical simulations. Lee and Hamada [18] investigated rupture propagation through sandy deposits under variable overburden densities. Lin et al. [19] studied overburden soil deformation induced by thrust fault slip, while Johansson and Konagai [20] correlated ground surface deformation in wet and dry soils with tunnel damage. Gazetas et al. [21] formulated design recommendations for dip-slip fault-foundation interactions. For shallow urban subways, Kun and Onargan [22] presented a case study of the Izmir metro tunnel using continuum modeling across fault zones. Oettle and Bray [23] examined fault propagation through previously ruptured soil [23]. Bazar et al. [24] conducted centrifuge and 3D numerical studies on reverse faulting interactions with continuous and segmental tunnels [25]. In rock tunneling, Zhao et al. [26] simulated TBM excavation in brittle rock near fault zones for the Brenner exploratory tunnel. Normal faulting interactions were evaluated by Liu et al. [27], who analyzed rupture propagation through loose strata and its impact on mountain tunnels. Kiani et al. [2] performed centrifuge tests on segmental shallow tunnels in alluvial deposits affected by normal faults. Ghalandarzadeh et al. [28] synthesized centrifuge data for geotechnical structures crossing active faults. Varnosefaderani et al. [29] analyzed cylindrical tunnels under near-fault seismic excitations combined with reverse faulting. Further geotechnical, foundation, and lifeline mitigation strategies were developed by Ashtiani et al. [30], Garcia and Bray [31], Manouchehrian and Cai [32], Yang and Mavroeidis [33], Yeganeh Khaksar et al. [34], Cai et al. [35], Tali et al. [36], and Wang et al. [37]. The era from 2020 to 2026 marked significant progress in shallow continuous tunneling, advanced limit analysis, and machine learning risk frameworks. Sabbagh and Ghalandarzadeh [38, 39] published benchmark centrifuge and 3D FE numerical studies on continuous shallow tunnels intersecting normal and reverse faults. Interaction mechanisms were expanded by Tohidifar et al. [40], Shen et al. [41], Agalianos and Anastasopoulos [42], Han and Li [43], Nabizadeh and Mojtahedi [44], and Yao et al. [45]. Mitigation mechanics using upper bound finite element limit analysis (UBFELA) were developed by Firouzeh et al. [46], Firouzeh et al. [47], and Nalkiashari et al. [48] for foundations crossing dip-slip faults. Su et al. [49] studied karst cave tunnel deformation, while Wang et al. [50] modeled reverse faulting impacts on mountain tunnels. Specifically regarding shallow continuous tunnels, Amini et al. [51] established numerical simulations modeling shallow continuous tunnels subjected to active dip-slip faults, providing detailed evaluations of longitudinal lining stresses, sinkhole formation, and overburden ratios (H/D). Amini and Khoshrou [52] integrated machine learning algorithms with non-linear numerical datasets to predict failure risks, structural vulnerability, and stress concentration zones for shallow continuous tunnels subjected to dip-slip faulting. Recent state-of-the-art studies by Aghamolaei et al. [53], Vasef et al. [54],

Wang et al. [55], Wang et al. [56], and Zhou et al. [57] highlight current engineering directions in fault-foundation interaction, resilient seismic design, and sensitivity analysis.

Despite substantial progress in previous experimental and numerical investigations, a key gap remains: the "mechanism-based, quantitative characterization of deformation transfer in segmental tunnel linings subjected to reverse-fault displacement". Previous studies have successfully established the importance of tunnel location, tunnel rigidity, fault geometry, overburden, lining properties, and soil conditions. However, the relative importance of these parameters and their influence on different components of the tunnel-ground response are not yet consistently ranked within a unified three-dimensional framework. In particular, the relationship between fault rupture propagation in the soil and the resulting longitudinal deformation, segment-ring rotation, and localized joint response requires further systematic investigation. Accordingly, the objective of the present study is not to re-examine the existence of reverse fault-tunnel interaction, which has already been established, but to provide an integrated experimental-numerical characterization of the deformation mechanisms of **segmental tunnels** under reverse-fault displacement. A centrifuge-based physical model is first used to assess the ground and tunnel response, followed by a three-dimensional finite-element model in ABAQUS [58] that is verified against the experimental observations. The numerical framework is then employed to systematically examine the effects of fault dip angle, soil stiffness, tunnel overburden, soil friction angle, and tunnel diameter on ground deformation, tunnel crown displacement, longitudinal S-shaped deformation, segment-ring rotation, joint response, and fault-zone propagation. In addition, a normalized sensitivity index (NSI) is introduced to quantitatively rank the relative influence of the investigated parameters within the adopted one-factor-at-a-time framework. The main contribution of this study is therefore the **quantitative and mechanics-based synthesis of how fault geometry, soil stiffness, burial depth, and tunnel geometry control the magnitude and localization of deformation in segmental tunnel systems**. Particular attention is given to distinguishing parameters that primarily attenuate global deformation from those that promote localized fault-induced deformation and modify the rupture-transfer mechanism around the tunnel. The resulting framework provides a systematic basis for interpreting the vulnerability of segmental tunnel linings under reverse-fault displacement and for identifying the parameters that should receive priority in seismic assessment and preliminary design.

2. Methodology

Laboratory modelling is generally performed using shake table modelling and geotechnical centrifuge modelling. In these methods, the desired prototype is downsized using scaling criteria, and the model is made at a small scale in the laboratory. Then, using a shake table or centrifuge, the desired acceleration is applied to the model, and the parameters are measured. This research project is a study conducted on the behaviour of buried structures, including lifelines, especially subway tunnels, where these structures pass through existing faults. This will be done by numerical modelling using finite elements in ABAQUS [58]. The results of centrifuge tests will also be used for verification.

2.1. Numerical model

In the present study, the soil environment has been studied using the softening Mohr-Coulomb constitutive model. The parameters of this constitutive model are:

ψ_p : final dilation angle, ψ_{res} : residual dilation angle, φ_p : final friction angle, φ_{res} : residual friction angle, and γ_f^p : octahedral shear strain rate in the final softening step.

In the present study, the parameters of the softening Mohr-Coulomb constitutive model, according to the study by Fadaee et al. [59] and considering the soil type in this paper (Firouzkouh sand #161), are considered as shown in Table 1.

Table 1. Values of softening Mohr-Coulomb model parameters in the present study.

Parameter	Value
ψ_p	12
ψ_{res}	1
φ_p	37
φ_{res}	32
γ_f^p	0.165

The softening Mohr-Coulomb model was adopted to represent the strain-softening behavior of Firouzkouh sand and the associated localization of shear deformation during reverse-fault displacement. The constitutive parameters used in the present study were adopted from the experimental and numerical investigation of Fadaee et al. [59], which specifically considered Firouzkouh No. 161 sand. The selection of these parameters is consistent with the density condition of the physical models, as the centrifuge soil beds were prepared at approximately 60% relative density, representing an intermediate density condition between the loose and dense Firouzkouh sand specimens investigated by Fadaee et al. [59]. However, these parameters should be regarded as literature-based representative parameters rather than as independently calibrated constitutive parameters for the specific stress paths developed during reverse-fault displacement in the present soil-tunnel system. In particular, the experimental program underlying the present study was not designed to provide stress-path-specific element-level constitutive data for independently calibrating the peak-to-residual strength degradation and the associated softening strain. Accordingly, the agreement between the numerical predictions and the centrifuge observations should be interpreted as system-level validation of the overall soil-tunnel response rather than as a

quantitative material-level calibration of the softening constitutive parameters.

Advanced constitutive formulations, such as the pressure-dependent multi-yield-surface (PDMY/PDMY02) models, provide a more comprehensive framework for representing nonlinear and stress-dependent soil behavior, particularly under cyclic loading and liquefaction-related conditions. Recent studies have demonstrated the application of PDMY02 in three-dimensional nonlinear analyses of liquefiable soil deposits and ground-improvement systems [60, 61]. In particular, Asgari and Forcellini [60] demonstrated the use of PDMY02 for different sand states and calibrated the corresponding parameters using field information and undrained cyclic simple-shear test data, highlighting the importance of stress-path-relevant experimental calibration when advanced constitutive models are employed. Asgari et al. [61] further demonstrated the application of PDMY02 within three-dimensional coupled hydro-mechanical analyses of liquefiable sandy soils and ground-improvement systems. These studies illustrate the potential advantages of advanced constitutive formulations while also emphasizing their dependence on sufficiently constrained laboratory and field data.

In the present study, a dedicated sensitivity analysis of the key softening parameters, particularly the peak-to-residual strength degradation and the softening strain, was not performed. Although a sensitivity analysis of the principal geometric and soil parameters is presented later in the manuscript, this analysis does not directly quantify the uncertainty associated with the softening parameters. Introducing arbitrarily selected alternative values for these parameters without independent experimental constraints could result in physically unsupported constitutive parameter combinations. Therefore, this limitation is explicitly acknowledged, and the numerical results are interpreted within the range supported by the adopted literature-based formulation. A systematic sensitivity analysis of the softening parameters, combined with stress-path-specific laboratory testing, is recommended for future work to quantify their effects on fault-rupture propagation, shear-band localization, and tunnel deformation.

2.2. Laboratory model

Soil behaviour is a function of its confining stress. An important point in geotechnical modelling is to equalize soil stress in the real model and the scale. The most important goal of the geotechnical centrifuge device is to eliminate the weakness of modelling in 1g environments due to the reduction of confining stresses in the soil. In other words, by downsizing the model to n times smaller in the laboratory, the all-round stresses are reduced by the same proportion in 1g modelling. In geotechnical centrifuge modelling, this weakness is compensated by imposing a centripetal force as a result of rotating the test sample around a fixed axis. Therefore, the sample rotation speed is adjusted so that the acceleration is equal to the ratio of the similarity of the large-scale to the small-scale model. A geotechnical centrifuge from Tehran University was used for the experiments in this research. This device is of the C67-2 model with a cantilever beam and a suspending basket, and its arm length is 3 meters. It is capable of accelerating between 5 and 130 times the acceleration of the Earth's gravity. To model the tunnel, asbestos-cement pressure pipes were used. This ensured the uniformity of the materials. The thickness determined by the similarity relations for the model tunnel was less than that of the produced pipes; however, the pipes were reduced in thickness after the turning operation, and the pipes were divided into 6 segments by cutting and drilling to form model tunnel rings. As the embankment aligned with the tunnel axis, the embankment was stopped, and the tunnel entered the soil (Fig. 1). After placing the tunnel, the embankment continued to reach the ground level as before. Fig. 2 shows the construction process of the segmental tunnel. For all experiments, 60%-density soil was used. The method for building the model was the wet compression method. In this method, 5% water was added to the dry sand, and a moist, homogeneous soil was obtained by mixing it. In the present study, the soil was poured into 3 cm layers and compacted by a tamper to achieve complete homogeneity throughout the soil reservoir. The number of poundings to reach the desired density in the pilot experiments was determined.

The numerical simulations were implemented using the ABAQUS/Explicit [62] solver to accurately capture the complex, highly non-linear interactions and the dynamic response during the fault displacement process. The computational domain was designed to be sufficiently large relative to the tunnel diameter (D) to minimize potential boundary interference. To model the soil medium, the domain was discretized using 8-node linear brick elements with reduced integration (C3D8R), which are specifically chosen to prevent volumetric locking and enhance computational efficiency in geotechnical problems. A refined mesh was strategically applied in the vicinity of the fault plane and the tunnel lining, where the maximum element size was optimized to ensure a clear representation of the plastic shear bands and stress gradients. The accuracy of this discretization was further verified through mesh convergence checks, ensuring that the maximum ground settlement and tunnel crown displacement reached stable values. Regarding the boundary conditions, the bottom of the model was fully constrained in all degrees of freedom, while the lateral boundaries were configured to allow vertical movement while preventing lateral translation, thereby simulating far-field soil behavior. The interaction between the segmented lining and the surrounding soil was governed by a surface-to-surface contact algorithm, utilizing a Coulomb friction law to permit realistic sliding and separation at the soil-lining interface. Furthermore, the segmented tunnel lining was modeled to account for its structural flexibility, allowing for the transfer of normal confining pressures and the development of localized relative rotation and shear displacement among the rings, which is critical for capturing the behavior of segmental linings under fault-induced deformations. The tunnel lining was modeled as a series of discrete segmental rings rather than as a continuous lining. Accordingly, adjacent rings were not assumed to behave as a continuously rigidly connected structure, allowing relative deformation between neighboring rings under fault-induced ground movement. The discrete representation permits localized relative rotation, shear displacement, and opening between adjacent rings, which are subsequently used to characterize the segmental response of the tunnel. The soil-lining interface was modeled using a surface-to-surface contact formulation with Coulomb friction, allowing relative sliding and separation between the surrounding soil and the tunnel lining. It should be noted that the present model does not employ an experimentally calibrated nonlinear moment-rotation constitutive law for the segmental

joints. Therefore, the calculated joint opening and ring rotation should be interpreted as indicators of the relative deformation of adjacent discrete rings rather than as a detailed constitutive prediction of joint moment-rotation behaviour.



Fig. 1. Images of the construction process of the segmental tunnel using asbestos-cement pipe.

Table 2 summarizes the similitude relationships adopted in the centrifuge modeling, where the scaling factors are defined as a function of the centrifugal acceleration level, N . In total, 32 experimental tests were performed, of which 18 were for normal fault and 14 for reverse fault. Of these, 20 were original experiments, and the rest evaluated the performance of the fault simulator box, model measurement tools, testing issues, and result repeatability. Table 3 provides a general overview of each experiment.

Table 2. Common coefficients in centrifuge modeling.

Parameters	Scale (model/prototype)
Length (diameter and thickness)	$1/N$
Density, stress, and strain	1
Velocity	1
Force (compressive and shear)	$1/N^2$
Bending moment	$1/N^3$
Young's modulus (E)	1
Flexural stiffness (EI)	$1/N^4$
Axial stiffness (EA)	$1/N^2$

Table 3. General overview of the experiments in the study.

No.	Fault	Fault angle	Acceleration (g)	Overburden/tunnel Diameter (H/D)	Internal diameter of prototype tunnel (m)
1	Reverse	60	60	1	6
2	Reverse	60	50	1	5
3	Reverse	60	50	0.75	5
4	Reverse	60	50	0.75	5
5	Reverse	60	50	0.75	5
6	Reverse	60	50	1	5
7	Reverse	75	50	0.75	5
8	Reverse	75	50	0.75	5
9	Reverse	75	50	1.3	5
10	Reverse	60	50	1	5
11	Reverse	60	50	1.5	5
12	Reverse	60	35	1	4.2
13	Reverse	60	35	Free filed	Free field
14	Reverse	60	35	0.75	4.2

In this section, one of the physical models created in a centrifuge device is selected to evaluate the accuracy of the numerical model. The numerical model was developed in ABAQUS [58] using the soil parameters reported by Fadaee et al. [59] for Firouzkuh No. 161 sand. The applicability of the adopted parameters within the present soil-tunnel configuration was subsequently assessed through comparison with the available centrifuge experimental results. In making the physical models, a box with an external diameter of $50 \times 60 \times 110$ cm is used, which is capable of modelling faults with angles of 45, 60, 75, and 90 degrees. It also considers the tunnel burial operation in its experiments between 0.75 and 1.5 times the tunnel diameter. The tunnel diameter in this modelling set is considered to be 4.2 m and 5 m. Fig. 2 illustrates the physical model.

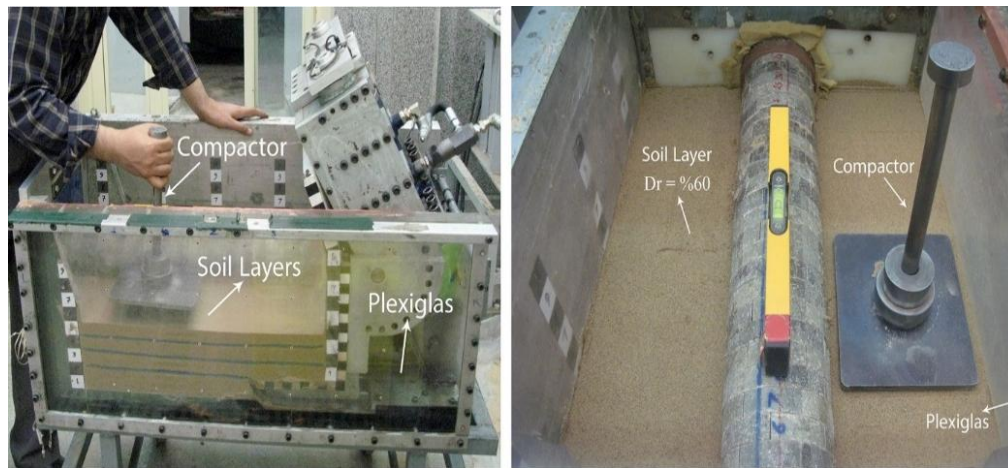


Fig. 2. The physical model.

The models selected for numerical model verification and assessment of modelling accuracy are models 2 and 21 of the physical models that have specifications as shown in Table 4.

Table 4. Geometric properties of physical models used for numerical verification.

Parameter	Model 2	Model 21
Tunnel diameter (D)	5 m	Free field
Overburden	1 D	Free field
Fault type	Reverse	Reverse
Fault angle	60	60
Centrifuge acceleration	50 g	35 g

To enhance the universality of the findings, the numerical results for tunnel displacements (u) and fault-induced deformation zones are normalized with respect to the tunnel diameter (D) and burial depth (H). This dimensionless approach (expressing as u/D and H/D) allows for a more comprehensive comparison of the structural interaction mechanism across different configurations, facilitating the application of these findings to tunnels with varying geometric scales.

3. Results

3.1. Verification of numerical model

To verify the numerical model results used in the analysis of the behaviour of these tunnels due to fault displacement near the fault zone, it is necessary to compare the results of numerical modelling with the experimental results on physical models and to ensure the accuracy of the numerical model. The laboratory results and numerical model are summarized in Table 5, and Fig. 3 also shows the parameters used for verification.

Table 5. Quantitative comparison between the experimental and numerical results for model 2.

Parameter	Laboratory model	Numerical model 2	Error (%)
Peak ground settlement (cm)	98	96	2%
End of tunnel crown level (maximum tunnel crown upward displacement) (cm)	+92	+88	4%
Beginning of the tunnel crown level (minimum tunnel crown downward displacement) (cm)	−5	−17	240%
Ratio of ground settlement to the end of tunnel crown level	1.06	1.09	3%
Differential tunnel crown vertical displacement (cm)	97	105	8%

Note: The error is calculated as the absolute difference between the numerical and experimental values divided by the absolute experimental value:

$$Error = \frac{|Max - Min|}{|Min|} * 100$$

A relatively large percentage difference of 240% is observed for the minimum settlement at the beginning of the tunnel crown level. However, this value should be interpreted with caution because the percentage error is calculated relative to the experimental value of −5 cm. The corresponding numerical prediction is −17 cm, resulting in an absolute difference of 12 cm. Thus, the large percentage value is primarily associated with the small magnitude of the experimental reference value and the sensitivity of relative error calculations to small reference values. This local discrepancy does not represent the overall performance of the numerical model. In fact, the remaining deformation parameters show close agreement between the experimental and numerical results, with errors of 2%, 4%, 3%, and 8% for the peak ground settlement, settlement at the end of the tunnel crown level, settlement ratio, and settlement difference, respectively. The overall consistency of these parameters indicates that the numerical model adequately captures the principal deformation characteristics of the tunnel-ground system. The larger discrepancy in the minimum crown settlement is therefore considered a localized difference that may be related to the sensitivity of the initial deformation state and local soil-tunnel interaction to the numerical idealization.

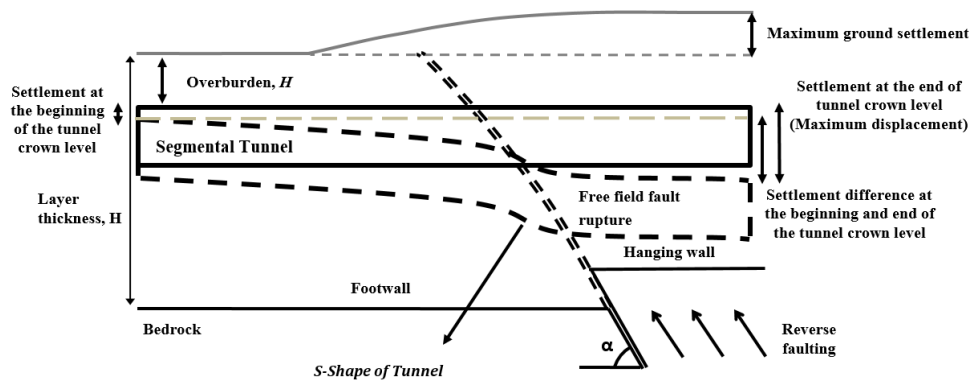


Fig. 3. Schematic of the modelled tunnel and the parameters affecting the model verification.

To verify the numerical modelling results, the distribution of the fault zone in the soil has been compared with experimental results. A comparison of the deformation in the tunnel rings is shown in Fig. 4. The visual comparison effectively demonstrates a strong agreement between laboratory observations and numerical simulations. The physical deformation of the segmental lining – specifically the joint opening and rotation – aligns closely with the structural response predicted by the Explicit numerical model. Furthermore, the correlation between shear-band propagation in the soil and the corresponding plastic shear strain contours provides additional evidence that the numerical framework captures the principal deformation mechanisms observed in the physical model. Overall, the comparison indicates that the adopted numerical framework can reproduce the main characteristics of the soil-tunnel response, including ground deformation, tunnel crown displacement, and localized deformation of the segmental lining. However, this system-level agreement should not be interpreted as an independent quantitative calibration of the adopted softening constitutive parameters.

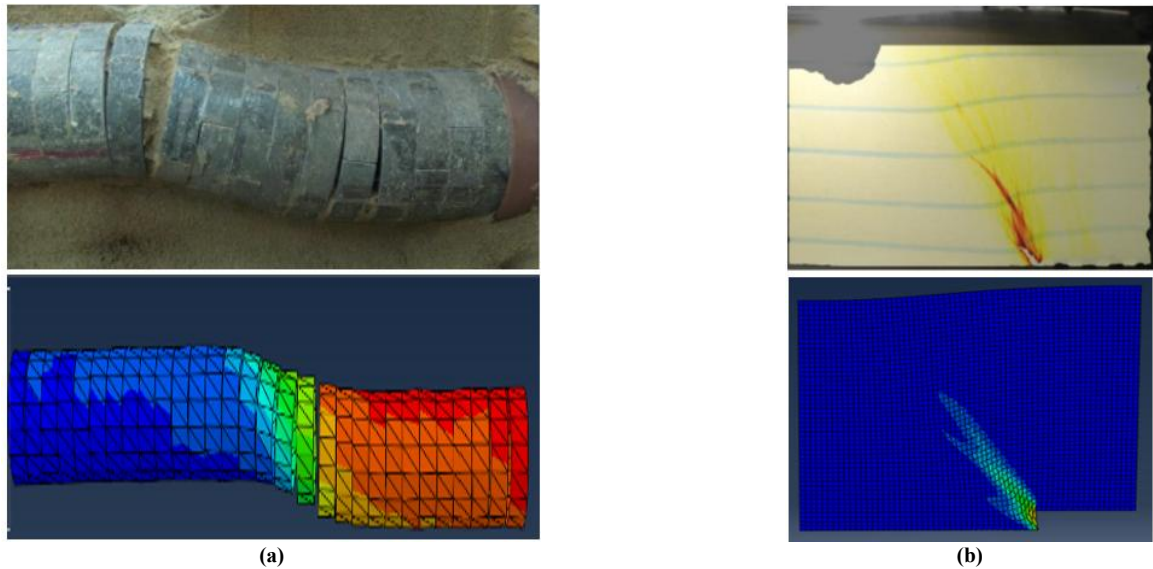


Fig. 4. (a) Deformation of tunnel rings in numerical model 2 with the Explicit method; and (b) numerical model and image analysis in model 21, Explicit method.

3.2. Parametric sensitivity analysis

A series of controlled one-at-a-time (OAT) sensitivity analyses was performed to investigate the influence of the principal geometric, loading, and soil parameters on the interaction between the reverse fault and the segmental tunnel (Table 6). The investigated parameters include tunnel diameter, fault dip angle, burial depth, soil elastic modulus, and friction angle. The OAT approach was deliberately adopted to isolate the effect of each parameter while maintaining the remaining parameters at their reference values. Therefore, the results should be interpreted as single-parameter sensitivity scenarios rather than as independently calibrated physical soil states.

To provide a quantitative comparison of the relative influence of the investigated parameters, a normalized sensitivity index (NSI) was introduced as Eq. 1.

$$NSI = \frac{\frac{\Delta Y}{Y_{ref}}}{\frac{\Delta X}{X_{ref}}} \quad (1)$$

where X denotes the investigated input parameter, Y represents the selected tunnel-response measure, and the subscript “*ref*” denotes the reference model. In the present analysis, the difference between the tunnel-crown displacements at the beginning and end of the tunnel was selected as the response measure because it directly characterizes the differential deformation induced by fault movement. The NSI therefore provides a dimensionless measure of the relative response variation associated with a normalized change in each input parameter. The calculated NSI values in Table 7 indicate that the fault dip angle has the strongest relative

influence on the selected tunnel response ($NSI = 0.434$). The soil elastic modulus and tunnel overburden exhibit intermediate sensitivity, with NSI values of approximately 0.102 and 0.113, respectively. Tunnel diameter has a comparatively lower influence ($NSI = 0.051$), whereas the independently varied soil friction angle produces a very limited response ($NSI = 0.017$). These results quantitatively support the observed trends in the individual parametric analyses. It should be emphasized that these sensitivity indices quantify the isolated influence of individual parameters and should not be interpreted as measures of genuine parameter interaction. In particular, the present OAT analysis does not allow direct estimation of interaction terms between soil stiffness, friction angle, tunnel geometry, burial depth, and fault dip angle. Such effects would require a factorial, response-surface, or other multi-parameter experimental design in which two or more variables are varied simultaneously. Accordingly, the term “interplay” is used only in a qualitative mechanistic sense, and the present results are explicitly distinguished from statistically quantified interaction effects.

Table 6. Characteristics of the studied models for parameter analysis (reverse fault).

Model No.	Tunnel diameter, meter (D)	Angle of friction (ϕ)	Overburden, meter (H)	Elasticity modulus, MPa (E)	Reverse fault angle (α)
1 ref. model	5	37	10	25-50, moderate soil	60
2	5	37	10	25-50, moderate soil	75
3	5	37	10	25-50, moderate soil	85
4	5	37	10	75-50, hard soil	60
5	5	37	10	12.5-25, soft soil	60
6	5	37	15	25-50, moderate soil	60
7	5	37	20	25-50, moderate soil	60
8	5	25	10	25-50, moderate soil	60
9	5	30	10	25-50, moderate soil	60
10	7.5	37	10	25-50, moderate soil	60
11	10	37	10	25-50, moderate soil	60

Table 7. Normalized sensitivity indices for the investigated parameters.

Parameter	Investigated range	Response range (cm)	Normalized sensitivity index (NSI)	Relative influence
Fault dip angle (α)	60-85°	177-209	0.434	High
Soil elastic modulus (E)	12.5-75 MPa	163-184	0.102	Moderate
Overburden depth (H)	10-20 m	158-178	0.113	Moderate
Tunnel diameter (D)	5-10 m	168-177	0.051	Low
Soil friction angle (ϕ)	25-37°	176-177	0.017	Very low

Note: The NSI represents the normalized single-parameter sensitivity within the adopted one-factor-at-a-time framework. The index is not an interaction coefficient and therefore does not quantify multi-parameter interaction effects.

Table 8 shows the displacement values created in the models. Also, Fig. 8 plots the distribution of faults in the soil for different models of reverse faults.

Table 8. Displacement values created in the numerical models.

Model No.	Ground vertical displacement (cm)	Beginning of the tunnel crown level (minimum tunnel crown downward displacement) (cm)	End of tunnel crown level (maximum tunnel crown upward displacement) (cm)	Ratio of ground settlement to the end of tunnel crown level	Differential tunnel crown vertical displacement (cm)
1	216	27	204	1.05	177
2	236	20	229	1.03	209
3	230	27	236	0.97	209
4	215	21	205	1.04	184
5	198	30	193	1.02	163
6	209	19	197	1.06	178
7	179	18	176	1.01	158
8	216	28	205	1.05	177
9	216	28	204	1.05	176
10	195	18	186	1.04	168
11	210	23	196	1.07	173

3.3. Investigation of the fault angle effect

In this paper, the behaviour of near-fault tunnels under three angles, 60, 75, and 85 degrees for reverse faults has been studied.

As shown in Fig. 5a, as the fault angle increases in a reverse fault, the maximum vertical displacement in the tunnel crown increases. By examining Fig. 5b, it can be seen that the maximum settlement created at ground level in a reverse fault increases by

20 cm, and it remains almost constant as the fault angle increases. As shown in Fig. 5c, by increasing the fault angle in the reverse fault from 60 to 75, longitudinal S-shaped tunnel deformation increases, and then this amount remains constant as the angle increases from 75 to 85.

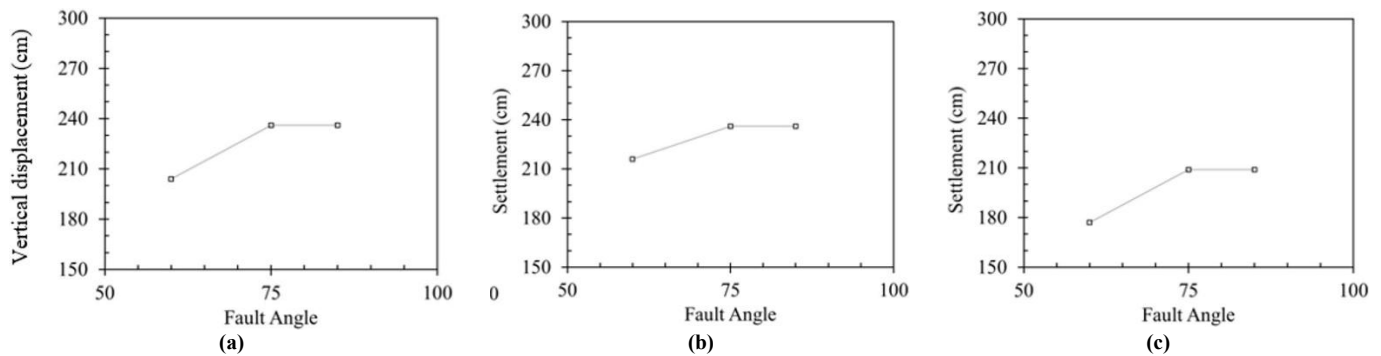


Fig. 5. Effect of fault angle on tunnel: (a) tunnel crown; (b) ground level; and (c) S-shaping of tunnel (tunnel diameter = 5 m, friction angle = 37°, moderate soil, and overburden = 10 m).

Fig. 6 illustrates the influence of the fault dip angle on the propagation of the rupture zone within the soil medium and the corresponding structural response of the tunnel lining under reverse faulting conditions. As observed, increasing the dip angle shifts the rupture pattern from a relatively planar shear band towards a distinct bilinear path, causing localized strain concentration near the ground surface and the tunnel crown. Furthermore, higher fault angles exacerbate the structural deformation of the tunnel lining. This is evidenced by a more pronounced S-shaped longitudinal deformation, coupled with severe relative rotation between adjacent segmented rings, which consequently leads to increased joint opening and potential loss of structural integrity.

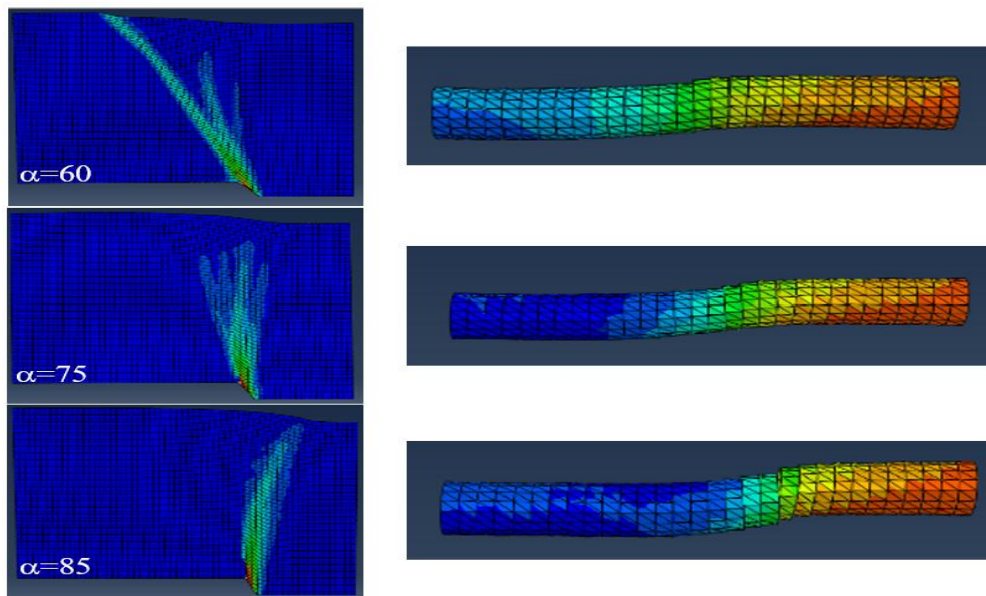


Fig. 6. Changes in fault distribution with fault angle variation under a reverse fault (tunnel diameter = 5 m, friction angle = 37°, moderate soil, and overburden = 10 m).

3.4. Investigation of the effect of soil elasticity modulus

The elastic modulus was varied from 12.5 to 75 MPa while the remaining soil, tunnel, and fault parameters were kept unchanged. This analysis was performed to isolate the influence of the stiffness level of the surrounding soil on fault-induced deformation and shear-band localization. The results indicate that increasing the soil elastic modulus reduces the overall deformability of the ground surrounding the tunnel and produces a more concentrated and continuous localized deformation zone. Conversely, lower stiffness results in a broader and more distributed deformation zone and greater tunnel deformation. These observations demonstrate the important role of ground stiffness in controlling the transfer and localization of fault-induced deformation around the tunnel.

By examining Fig. 7a, it can be seen that the amount of vertical displacement created at the tunnel is similar to that of the subsidence at ground level and has an optimum value for the soil elasticity modulus. Therefore, as the hardness of the soil increases, the amount of subsidence created in the vertical direction due to gravity decreases; at the same time, due to complete transfer of bedrock displacement to the whole soil, the displacement resulting from the fault increases. In this study, the behaviour of near-fault tunnels placed in three types of soft, moderate, and solid soils in a reverse fault has been investigated. The following are the diagrams for each. As shown in Fig. 7b, as soil type changes from soft to moderate in reverse faulting, the amount of displacement created at the surface decreases because of increasing soil hardness, and then this value remains constant. Therefore, it can be said that the elasticity modulus of soil has an optimum value, and an increase in soil hardness from a certain amount will not have any effect on the amount of displacement created at ground level. As shown in Fig. 7c, with increasing soil hardness in reverse faults, the opposite is true, and as hardness increases, the displacement difference at the beginning and end of the tunnel crown level increases.

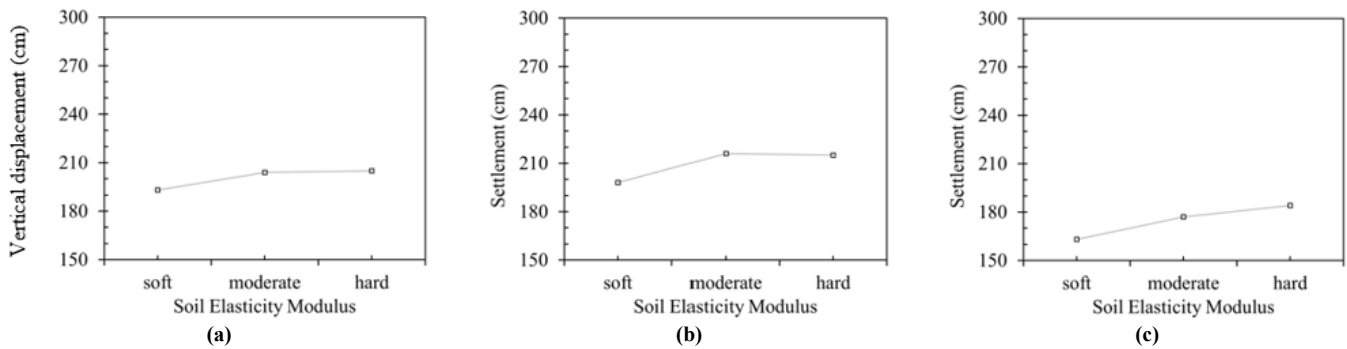


Fig. 7. Effect of soil type on tunnel: (a) tunnel crown; (b) ground level; and (c) S-shaping of tunnel (tunnel diameter 5 m, friction angle 37° , fault angle 60° , and overburden 10 m).

As illustrated in Fig. 8, increasing soil stiffness (elastic modulus) significantly influences both the rupture band propagation within the soil medium and the structural behavior of the tunnel lining under reverse faulting. In softer soils, the shear zone tends to diffuse and branch out with irregular patterns, partially attenuating before reaching the surface. Conversely, as the soil stiffness increases from soft to hard, the rupture plane develops fully and propagates continuously up to the ground surface along a well-defined, localized shear band with minimal branching. Regarding the structural response of the tunnel, higher soil stiffness induces greater soil-structure interaction constraints, exacerbating the localized shear forces on the lining. Consequently, a more pronounced S-shaped longitudinal deformation is observed in stiffer ground conditions. Furthermore, the increased ground resistance intensifies the relative displacement and rotation between adjacent segment rings, leading to a substantial increase in joint opening and joint dislocation.

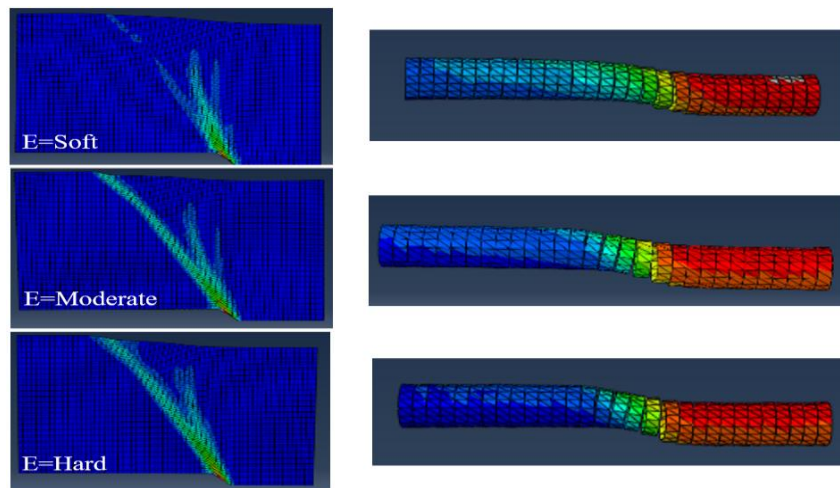


Fig. 8. Changes of fault distribution with changes in soil hardness in a reverse fault (tunnel diameter = 5 m, friction angle = 37° , fault angle = 60° , and overburden = 10 m).

3.5. Investigation of the effect of overburden

To investigate the effect of tunnel depth, three values, 10, 15, and 20 m, are considered for tunnel overburden, and then the results for reverse faults are investigated. It is observed that as the tunnel depth increases, the amount of displacement created at the surface for reverse faults decreases. The notable point is that the slope changes with an increase in tunnel overburden. The amount of displacement created at the tunnel crown level is similar to the surface behaviour according to Fig. 9, and changes increase as slope depth increases. This may be due to the increase in soil mass due to the increase, and consequently a decrease in the fault force for displacement across the whole section of soil.

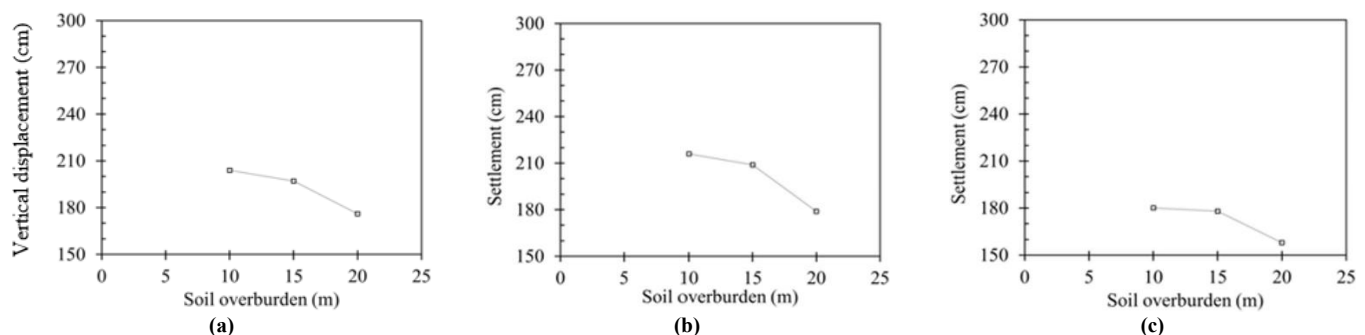


Fig. 9. Effect of overburden on tunnel: (a) tunnel crown; (b) ground level; and (c) S-shaping of tunnel (tunnel diameter = 5 m, friction angle = 37° , moderate soil, and fault angle = 60°).

As shown in Fig. 9, by increasing the tunnel overburden in a reverse fault, the displacement difference at the beginning and end of the tunnel crown decreases because of an increase in tunnel depth.

As illustrated in Fig. 10, the overburden depth plays a critical role in controlling both the fault rupture propagation through the soil and the structural deformation of the tunnel lining under reverse faulting. With increasing overburden depth (from $H = 10$ m to 20m), the confinement around the tunnel increases significantly, which alters the soil-structure interaction mechanisms. As a result of higher confinement at greater depths, the tunnel experiences higher shear forces, leading to a more pronounced S-shaped longitudinal bending deformation. Conversely, the increased surrounding soil pressure suppresses excessive relative movement between adjacent segment rings, thereby reducing the joint openings (gaps) and ring offsets compared to shallow overburden conditions. From a mechanical perspective, the observed reduction in crown displacement with increasing overburden (H/D ratio) is attributed to the enhanced soil confinement. As the overburden increases, the lateral earth pressure coefficient and the resulting normal stress state around the lining rise significantly. This increase in confinement limits the rotational degrees of freedom at the segmental joints, thereby reducing the joint opening magnitude. Furthermore, the higher overburden acts as a stiffness buffer, effectively attenuating the fault-induced shear strain before it propagates to the lining interface. The internal force analysis confirms that under higher overburden, the bending moments (M) are redistributed more uniformly along the tunnel perimeter, preventing localized stress concentrations that typically lead to larger displacements.

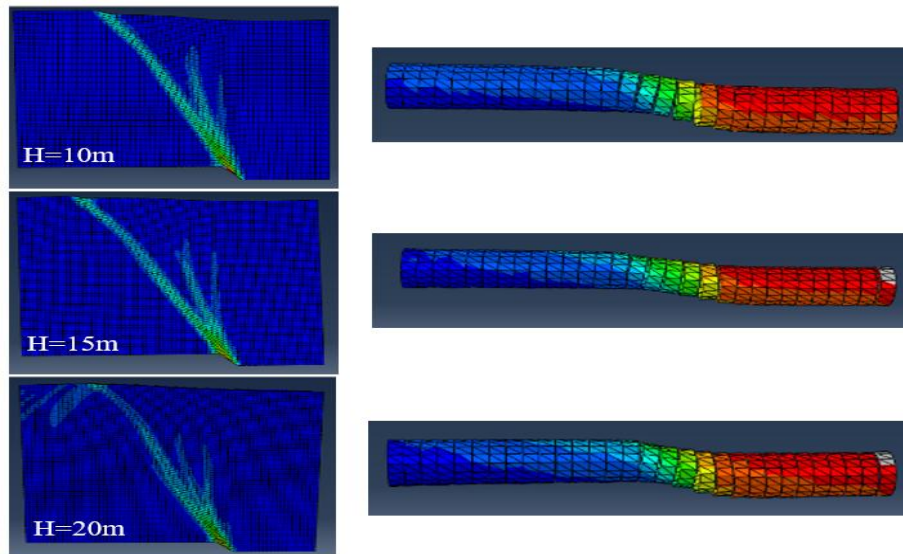


Fig. 10. Fault distribution changes with increasing overburden under a reverse fault (tunnel diameter = 5 m, friction angle = 37° , moderate soil, and fault angle = 60°).

3.6. Investigation of the effect of friction angle of soil grains

In the present study, this parameter was changed independently, and with other soil parameters kept constant, it was observed that the parameter did not change the numerical modelling results independently. The following is an example of a diagram of the maximum vertical displacement created at the tunnel crown level by reverse faulting in Fig. 11.

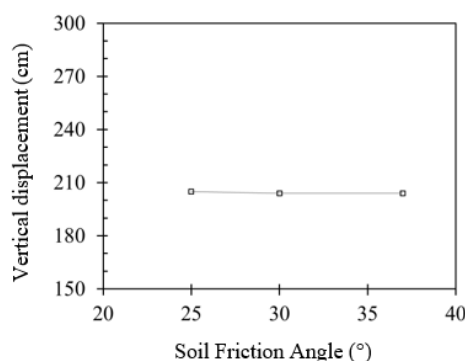


Fig. 11. Effect of friction angle on maximum vertical displacement at tunnel crown level (tunnel diameter = 5 m, moderate soil, overburden = 10 m, and fault angle = 60°).

By examining Fig. 11, it can be seen that the maximum vertical displacement created at tunnel crown level is not independently dependent on the friction angle of the soil. To study the effect of this parameter, its effect on other parameters of the soil behavioural model should be considered. Mere increasing the friction angle while keeping other parameters constant has very little effect on the fault-affected area and slightly changes the thickness of that area. It can also be seen that the variability of the mere friction angle of soil grains does not affect the S-shaping value and the tunnel ring orifice.

3.7. Investigation of the effect of tunnel diameter

In this study, three diameters, 5, 7.5, and 10 m, are modelled and studied for the tunnels. By increasing the diameter in reverse faulting, the displacement initially decreases and then increases (it has a relative minimum). According to Fig. 12, the behaviour of the tunnel is similar to that of the earth's surface. It is also seen from Fig. 12c that the effect of tunnel diameter on its S-shaping is

negligible, with a 50% increase in tunnel diameter only causing a 9 cm decrease and then a 5 cm increase in the S-shaping value of the tunnel.

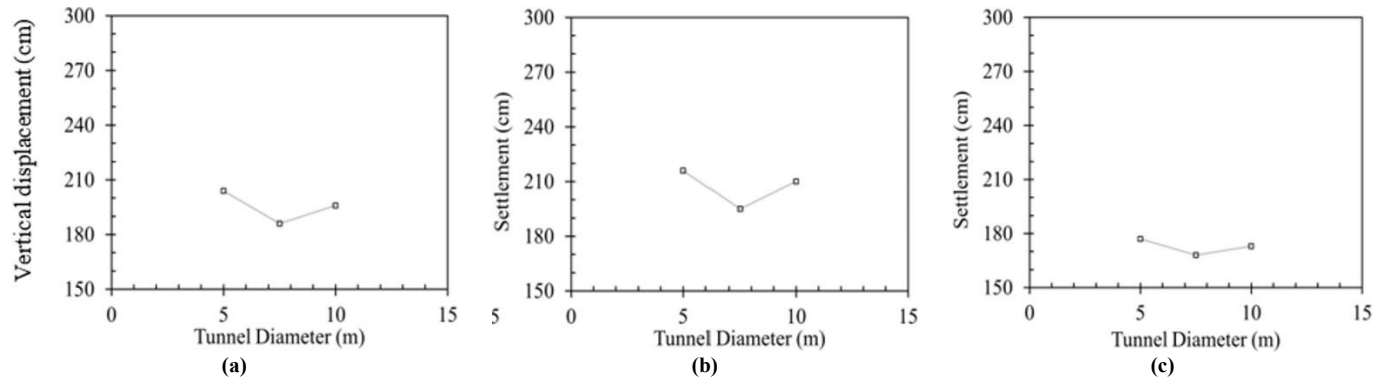


Fig. 12. Effect of tunnel diameter: (a) tunnel crown; (b) ground level; and (c) S-shaping of tunnel (friction angle = 37°, moderate soil, overburden = 10 m, and fault angle = 60°).

Fig. 13 illustrates the influence of tunnel diameter (D) on shear band propagation within the soil medium under reverse fault movement. As observed, increasing the tunnel diameter significantly alters the rupture propagation path due to the enhanced stiffness and blockage effect provided by the larger tunnel structure. In smaller diameters ($D = 5$ m), the primary rupture path easily propagates upward toward the hanging wall block and reaches the ground surface along a continuous shear zone with secondary vertical branches. However, as the tunnel diameter increases to $D = 10$ m, the larger cross-section acts as a rigid barrier, redistributing localized strains around the tunnel boundary. Consequently, rupture transfer into the hanging wall side is substantially suppressed, resulting in a more localized and constrained shear zone around the tunnel crown and primary fault trace. The influence of tunnel diameter on fault-induced deformation is governed by the structural flexural rigidity (EI). While global surface settlement remains relatively insensitive to diameter variations, the tunnel's structural response is highly dependent on the D parameter. Increasing the diameter enhances the tunnel's cross-sectional moment of inertia, which initially decreases the magnitude of differential displacements across the fault plane. However, the subsequent increase in displacement for larger diameters observed in our model is linked to the larger contact area with the soil, which facilitates the transmission of shear forces. A mechanics-based assessment of the axial force and bending moment profiles reveals that larger diameters induce a more complex S-shaped bending pattern, as the tunnel lining experiences higher differential loading across the fault-soil interface. These observations suggest that fault-displacement transfer is influenced not only by tunnel geometry but also by the relative stiffness-to-diameter characteristics of the tunnel lining. However, this observation represents a mechanistic interpretation of the single-parameter analysis rather than a direct quantification of parameter interaction.

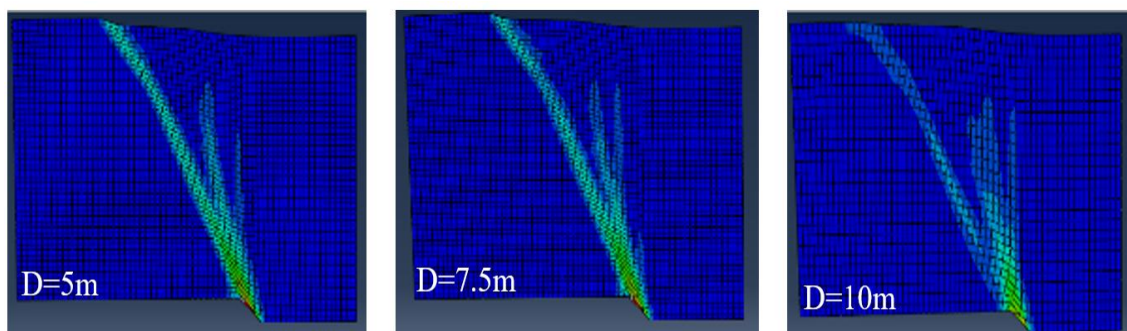


Fig. 13. Changes in fault distribution with increasing tunnel diameter under reverse fault (friction angle = 37°, moderate soil, overburden = 10 m, and fault angle = 60°).

4. Limitations and future research

The present study provides a detailed investigation into the interaction between reverse faults and segmental tunnel linings. While the deterministic approach offers high-fidelity insights into the deformation mechanisms, the following limitations should be noted:

- **Scope of fault configurations:** This study focuses on a specific reverse-fault geometry and displacement magnitude. The interaction mechanism – specifically the transition between structural bending and shearing – is highly dependent on the fault dip angle and the tunnel-fault crossing angle. Future parametric studies should investigate a broader range of crossing angles (e.g., 30° to 90°) and fault types, including strike-slip and normal faults, to derive more universal design charts.
- **Geotechnical and environmental factors:** The current model assumes a specific soil density and drainage condition. Variations in groundwater pressure and soil hydraulic conductivity can significantly alter the pore-water pressure response during fault movement, which should be incorporated into future constitutive modeling.
- **Scaling relationships:** The validity of our centrifuge model results relies on the application of appropriate scaling laws. We verified our prototype behavior by ensuring the similarity in stiffness-to-geometry ratios (EI/D^4) and soil-lining interface

friction. However, we acknowledge that the scaling relationship between centrifuge fault displacement and real-field prototype behavior is non-linear. Future work will involve a series of comparative simulations to refine the scaling factors for different lining stiffnesses.

5. Conclusions

This study investigated the behavior of segmental tunnels subjected to reverse fault displacement through an integrated experimental and numerical approach. Centrifuge modeling was first employed to reproduce the complex soil-tunnel interaction under reverse faulting, and the developed finite element model was subsequently verified against the experimental observations. The close agreement between the numerical predictions and centrifuge test results confirmed the capability of the adopted numerical model to accurately capture the deformation mechanisms associated with fault rupture, providing a reliable framework for conducting comprehensive parametric analyses. The results demonstrated that tunnel response is governed by the combined influence of fault geometry, soil mechanical properties, and tunnel burial conditions rather than by any individual parameter. Among the investigated variables, the reverse fault angle was identified as one of the most influential factors controlling tunnel performance. Increasing the fault angle intensified ground settlement, tunnel crown displacement, and S-shaped tunnel deformation while simultaneously enlarging the fault-affected zone and increasing tunnel ring rotation. These observations indicate that steeper reverse faults generate more concentrated deformation fields and consequently impose greater structural demands on segmental tunnel linings. The mechanical properties of the surrounding soil also played a significant role in determining tunnel behavior. Increasing soil stiffness reduced the magnitude of ground settlement due to the enhanced resistance of the soil mass against vertical deformation. However, stiffer soils transferred bedrock displacement more efficiently toward the ground surface, resulting in clearer and more continuous fault rupture propagation and greater localized deformation around the tunnel. Therefore, although hard soils may reduce overall settlement, they can increase the concentration of fault-induced deformation and should be carefully considered during seismic design. Tunnel burial depth considerably influenced both the magnitude and distribution of deformation. Increasing the overburden reduced surface settlement and tunnel displacement because the thicker soil cover dissipated a larger portion of the imposed fault movement before it reached the ground surface. Nevertheless, deeper tunnels exhibited more pronounced S-shaped deformation patterns, indicating that increasing burial depth modifies the deformation mechanism rather than eliminating the adverse effects of reverse faulting.

The parametric analyses further showed that varying the soil friction angle alone produced only a marginal effect on tunnel response when other constitutive parameters were maintained constant. This finding suggests that the friction angle cannot be considered an independent governing parameter and should instead be evaluated together with the complete constitutive behavior of the surrounding soil. Likewise, tunnel diameter exhibited only a limited influence on global settlement; however, increasing the tunnel diameter altered the distribution of plastic strains above the tunnel and modified the upward propagation of the fault rupture. Therefore, although tunnel diameter has a relatively minor effect on overall deformation, it may significantly influence the local failure mechanism within the fault zone. From an engineering perspective, the outcomes of this research emphasize that the design of tunnels crossing active reverse faults should explicitly consider the combined effects of fault angle, soil stiffness, and tunnel overburden. The numerical model verified against the centrifuge observations provides an efficient tool for evaluating tunnel performance under different geological conditions and can support engineers during the preliminary design and risk assessment of underground lifeline infrastructures located in fault-prone regions. Furthermore, the findings suggest that mitigation strategies such as adopting flexible segmental linings, improving joint ductility, optimizing tunnel alignment with respect to the fault trace, and applying localized ground improvement techniques may substantially reduce structural damage caused by reverse fault displacement. Beyond geometric and structural design modifications, advanced strengthening techniques may provide additional opportunities for improving structural resilience under severe deformation demands. Recent experimental research has demonstrated that prestressed fiber-reinforced polymer (FRP) confinement can enhance the load-carrying capacity, initial stiffness, and deformability of damaged reinforced-concrete members compared with conventional passive confinement [63]. Although this strengthening approach has primarily been investigated for reinforced-concrete members rather than tunnel linings subjected to fault displacement, its active confinement mechanism provides a potentially relevant concept for future tunnel retrofit applications. In particular, prestressed FRP-based strengthening may be investigated as a potential future technique for controlling localized damage and enhancing the residual structural capacity of tunnel lining components following severe deformation. Further experimental and numerical studies are required to evaluate its applicability to segmental tunnel linings under fault-induced deformation. Although the proposed methodology provides reliable predictions of tunnel behavior under reverse faulting, the present study was limited to specific soil conditions, tunnel geometries, and quasi-static fault displacement. Future investigations should extend the proposed framework to three-dimensional numerical analyses, dynamic earthquake loading, nonlinear segmental joint behavior, groundwater effects, and different geological environments. Incorporating probabilistic risk assessment and performance-based design methodologies would further improve the reliability and practical applicability of underground structures constructed across active fault zones.

Although the present study focuses on the deformation response of segmental tunnels subjected to reverse-fault displacement, the proposed fault-tunnel framework can be extended in future studies to investigate time-dependent and impulsive loading conditions. In this context, analytical studies of transient structural response, such as the dynamic response of a transverse beam under slamming excitation based on Wagner theory [64], provide a useful perspective for interpreting dynamic deformation transfer and structural demand. Such analytical approaches may offer a complementary basis for understanding the temporal evolution of deformation and internal forces in tunnel linings subjected to rapidly varying ground or fault-induced loading. Future investigations could therefore integrate transient dynamic formulations with the present fault-tunnel framework to evaluate the effects of loading

duration, excitation characteristics, and structural inertia on tunnel response.

Statements & declarations

Author contributions

Majid Kiani: Conceptualization, Methodology, Software, Validation, Formal analysis, Investigation, Writing - Original Draft, Writing - Review & Editing, Supervision.

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Data availability

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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Development and Application of a Simplified Multi-Criteria Framework for Soil Liquefaction Assessment: A Case Study

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ABSTRACT

The liquefaction phenomenon has been one of the most important causes of damage to structures and vital lifelines during past earthquakes. The accurate assessment of liquefaction has always faced multiple challenges. Common methods based on field data such as Standard Penetration Test (SPT), Cone Penetration Test (CPT), and Shear wave velocity (V_s) often provide different results, and relying on only one method can lead to errors in identifying liquefied layers, post-liquefaction settlement values, and the likelihood of surface manifestation. In this study, the evaluation of the safety factor was examined by four common methods. Moreover, three methods were employed to estimate the post-liquefaction settlement, and two methods were used to assess surface manifestation. The results indicate significant differences among the methods in determining the safety factor, the surface manifestation, and the settlement amount. Since none of the methods alone can fully capture the complexity of soil behavior, a four-step framework including multi-criteria assessment of liquefaction potential, settlement, and surface manifestation was introduced, and then the integration of results was presented. This approach, by reducing uncertainty and simultaneously considering three main components of liquefaction (occurrence potential, settlement, and surface manifestation), provides a potentially useful and more comprehensive engineering-oriented framework for assessing liquefaction-related hazards and can serve as a suitable basis for engineering studies and structural design in liquefaction-prone areas.

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Nomenclature & Abbreviations

Symbol	Description	Symbol	Description
D_r	Relative density	q_c	Cone tip resistance
D_{50}	Mean particle diameter	q_{c1}	Normalized cone tip resistance
FC	Fines content	Rf	CPT friction ratio
F_L	Liquefaction potential factor	S	Post-liquefaction settlement/displacement
H_1	Thickness of the non-liquefied surface layer	V_s	Shear-wave velocity
H_2	Thickness of the underlying liquefied layer	$W(z)$	Depth-dependent weighting function
$(N_1)_{60}$	Corrected SPT blow count	z	Depth of the considered soil layer
$(N_1)_{60cs}$	Fines-corrected SPT blow count for clean sand	γ	Soil unit weight
P_L	Liquefaction potential		

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Abbreviation	Description	Abbreviation	Description
AI	Artificial intelligence	K-NN	K-nearest neighbor
CPT	Cone penetration test	LSTM	Long short-term memory
CRR	Cyclic resistance ratio	PGA	Peak ground acceleration
CSR	Cyclic stress ratio	PLHA	Probabilistic liquefaction hazard analysis
FS	Factor of safety	SPT	Standard penetration test
GRU	Gated recurrent unit	SVM	Support vector machine

1. Introduction

The destruction of cities, casualties, and financial losses resulting from earthquakes are often due to settlement caused by liquefaction [1-3]. The liquefaction phenomenon has been observed in many earthquakes such as Niigata 1964, Alaska 1964, Manjil-Rudbar 1990, Kobe 1995, Chi-Chi 1999, Tohoku 2011, Christchurch 2011, Kumamoto 2016, Sulawesi 2018, Jishishan 2023, etc. [4-9]. The most important damaging effects of liquefaction include loss of bearing capacity, generation of buoyant forces, increased pore water pressure, sand boiling, lateral spreading, settlement, and heave [8]. For the first time, Casagrande [10] introduced the phenomenon of sand liquefaction and its effects. The term “spontaneous liquefaction” was used by Terzaghi and Peck [11] to describe the state change of loose sandy deposits and their flow like a viscous fluid due to slight disturbance. Studies show that numerous geotechnical parameters affect the occurrence of liquefaction, such as grain size distribution, initial relative density, drainage conditions, formation duration and dimensions of sand deposit, magnitude and nature of overburden, soil structure, prior deformation history, trapped air bubbles between particles, etc. [12-15]. Liquefaction has been a major cause of failure in soil structures, facilities, vital lifelines, and building foundations during past earthquakes [16-19]. Therefore, a complete understanding of liquefaction is essential for mitigation and improvement of structures and also for identifying liquefaction-susceptible zones.

The liquefaction potential depends on two factors: seismic load (intensity and duration) and the susceptibility of materials to liquefy. Various assessment methods have been proposed, including experimental-empirical and numerical methods. Experimental-empirical methods such as cyclic stress and cyclic strain approaches are popular due to their simplicity, acceptable accuracy, and computational speed [1, 8]. In the 1970s, significant advances in understanding the liquefaction phenomenon were made based on research by Seed and Idriss [20], focusing on defining the cyclic shear stress necessary for liquefaction, known as the cyclic stress method. Because many factors influence cyclic shear stresses, experimental evaluation of liquefaction resistance using cyclic stress alone is difficult. Hence, methods using cyclic strain instead of cyclic stress were suggested to define earthquake loading and liquefaction resistance. Since loading and resistance are determined based on strains instead of stresses, the cyclic strain method cannot provide a factor of safety (FS) against liquefaction. The advantage of the cyclic strain method is that it uses the strong relationship between pore pressure generation and cyclic strain amplitude. For a given soil, excess pore water pressure can be obtained more accurately from cyclic strain than cyclic stress. However, estimating cyclic strain is considerably more difficult than cyclic stress. Thus, cyclic strain has not become as common in seismic geotechnical engineering applications as cyclic stress [8].

Numerical methods have been developed to overcome weaknesses of experimental-empirical methods. New probabilistic methods such as probabilistic liquefaction hazard analysis (PLHA) use Monte Carlo simulations to consider statistical distributions of soil parameters and seismic accelerations, reducing uncertainty in liquefaction modeling. However, selection of an improper statistical distribution or seismic modeling assumptions can still cause error in risk prediction. Systematic reviews of PLHA reveal variability in models, logic tree construction, and seismic scenario selection lead to highly variable liquefaction hazard results. In other words, even among probabilistic models, differences in conceptual foundations, input data, and decision tree structure can limit the accuracy of liquefaction hazard prediction. Additionally, uncertainty in soil parameters and seismic models (particularly earthquake magnitude distribution and recurrence intervals) significantly affects liquefaction hazard maps, alongside numerical complexity and time consumption [7, 16, 21].

Recently, machine learning and deep learning methods have emerged to complement or replace empirical methods but face challenges. Artificial intelligence (AI) models such as support vector machine (SVM), k-nearest neighbor (K-NN), and deep networks such as long short-term memory (LSTM) and gated recurrent units (GRU) show high accuracy in predicting liquefaction sensitivity but depend heavily on the quality and preprocessing of experimental data. Prediction errors may arise due to overfitting or under-representation of uncertainty, particularly with limited training samples or unrepresentative data. Some studies have preferred to use interpretable models for predicting liquefaction because these models can help engineers understand which soil or seismic features play the most significant role in liquefaction hazard. However, this approach is limited by model assumptions and also it is sensitive to input data [16-22].

Evidence from reviews, case studies, and data-based research shows that no single method can perfectly predict liquefaction without error. A multi-method framework considering parameter, model, and input uncertainty is necessary for reliable prediction, improving confidence for engineering design and risk management. In routine engineering projects, the cyclic stress method is used to estimate liquefaction potential and provide safety factors at various depths, but this approach lacks accuracy and is subject to error. Most of the liquefaction assessment methods rely on field tests such as standard penetration test (SPT), cone penetration test (CPT), and Downhole of which uncertainties are now evident. Therefore, relying on one method that depends only on a single field test is not an appropriate approach. Furthermore, in the engineering projects, the evaluation only provides an FS regarding the occurrence or non-occurrence of liquefaction. In fact, two very important aspects-the surface manifestation of liquefaction and the

settlement caused by the liquefied layer-are not addressed in the current practice.

This study proposes a practical and simple approach to assess liquefaction potential in a borehole using various experimental-empirical methods, including those by Iwasaki et al. [23], Andrus and Stokoe [24], Moss et al. [25], and Boulanger and Idriss [26], based respectively on SPT, V_s , CPT, and SPT. These methods were chosen for their common use and acceptance. Besides, Tokimatsu and Seed [27], Ishihara and Yoshimine [28], and Wu [29] methods were utilized to assess seismic-induced settlement. Surface manifestation is evaluated using Iwasaki et al. [30] and Ishihara [31] methods. By comparing results, a simple multi-criteria practical framework is presented for liquefaction potential assessment in typical civil engineering projects.

It should be noted that while the authors have previously compared liquefaction assessment methods based on three field tests in a case study in Babol [1], the present study offers several novel contributions beyond that earlier work. First, the current study incorporates four liquefaction triggering methods instead of three, thereby providing a more comprehensive comparison. Second, unlike the previous study, which focused solely on the factor of safety, the present work systematically evaluates post-liquefaction settlement using three different field-test-based methods [27-29] and surface manifestation using two approaches [30, 31]. Third, and most importantly, this study introduces a practical four-step multi-criteria framework that integrates liquefaction triggering, settlement, and surface manifestation into a unified engineering classification. This framework represents a significant advancement over conventional single-method or single-criterion assessments and provides a structured basis for engineering decision-making in liquefaction-prone areas

2. Methods for evaluating liquefaction potential

Most experimental-empirical methods for assessment of liquefaction potential have been analyzed in Japan and the USA. Each of these methods is based on different assumptions and evaluates the liquefaction potential using various field test data. Below, the methods used in this study are briefly introduced. It is worth mentioning that this research employs methods that, first, are based on different field tests, and second, enjoy high acceptance within the engineering community.

2.1. Iwasaki et al. method

The process of evaluating liquefaction in this method is based on estimating the equivalent cyclic shear stress, which is likely to occur in the soil mass during an earthquake, as well as calculating the liquefaction resistance, and finally comparing them. It is also worth mentioning that this method is based on SPT results. The method proposed by Iwasaki et al. [23] was also adopted by the Japan Highway Bridge Association.

2.2. Andrus and Stokoe method

By measuring the shear-wave velocity (V_s) corresponding to small strains in the field, Andrus and Stokoe [24] introduced a new method for the determination of liquefaction resistance. The use of shear-wave velocity is based on the fact that both shear-wave velocity and the cyclic resistance ratio (CRR) are similarly affected by porosity ratio, effective confining stress, stress history, and geological age of the deposit. This method is applicable in soils that are difficult to sample, where penetration tests are unreliable, and at sites where deep drilling or probing is not feasible.

2.3. Moss et al. method

Moss et al. [25] method is based on detailed CPT results. Some advantages of this method include: collecting very extensive databases of case histories. Utilizing refined knowledge and experience regarding factors affecting CPT, site seismicity (directivity effects, site response, etc.), and cyclic stress ratio (CSR). Organizing case history data according to quality/uncertainty. Using higher-level probabilistic tools.

Moss et al. [25] method has not only significantly reduced uncertainty levels but also helped resolve several debated issues such as fines content correction and effective overburden stress correction. One key difference of this approach compared to others is calculating the stress reduction factor with much larger datasets.

2.4. Boulanger and Idriss method

Boulanger and Idriss [26] reviewed and updated semi-empirical methods for evaluating the liquefaction potential of saturated cohesionless soils during earthquakes for practical use. The updated items include the shear stress reduction factor, magnitude scaling factor, overburden correction factor for cyclic stress ratios, and overburden correction factor for in-situ tests. It is worth noting that this method is based on SPT results.

3. Methods for evaluating post-liquefaction settlement

In this section, the common methods used to evaluate post-liquefaction settlement will be introduced.

3.1. Tokimatsu and Seed method

Laboratory experiments have shown that volumetric strain varies with relative density and maximum shear strain. Tokimatsu and Seed [27] presented a relationship between the SPT number and CSR to estimate post-liquefaction strain for an earthquake with

a magnitude of 7.5 (Fig. 1). For earthquakes of different magnitudes, CSR can be determined by estimating CSR and adjusting it with a magnitude scaling factor.

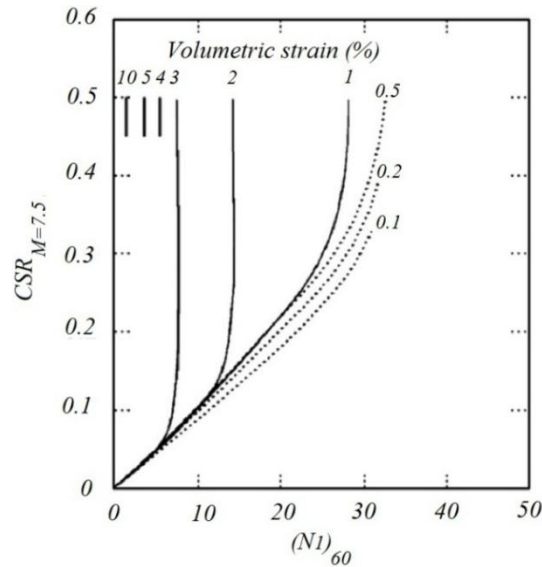


Fig. 1. The volumetric strain estimation based on the studies by Tokimatsu and Seed [27].

3.2. Ishihara and Yoshimine method

Ishihara and Yoshimine [28], continuing the efforts of Tokimatsu and Seed [27] and based on an extensive set of simple shear tests, proposed a method for predicting post-liquefaction settlements. This method expresses the volumetric strain as a function of relative density and also as a function of the FS against liquefaction (instead of CSR). It is worth noting that relative density can be estimated based on either of the two in-situ strength indices, namely the standardized SPT number or the cone tip resistance in CPT (Fig. 2).

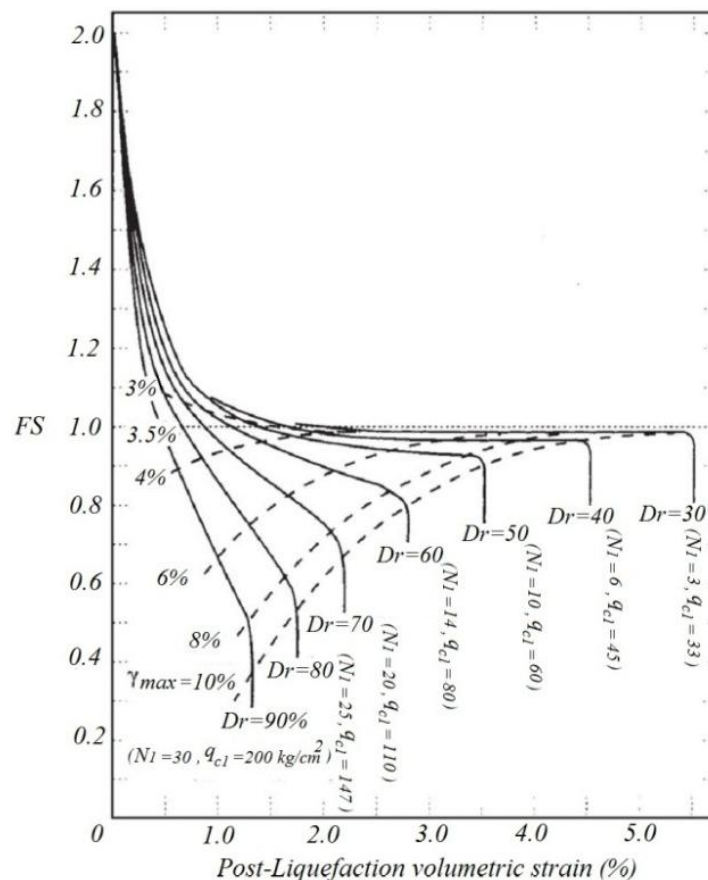


Fig. 2. The volumetric strain estimation based on the studies by Ishihara and Yoshimine [28].

3.3. Wu method

Wu [29] studied a collection of high-quality laboratory test databases and conducted a systematic test program to investigate the impact of relative density and effective stress on the generation of post-liquefaction shear and volumetric strains. He presented some curves, as a function of CSR and the corrected SPT number, to estimate post-liquefaction volumetric strain (Fig. 3).

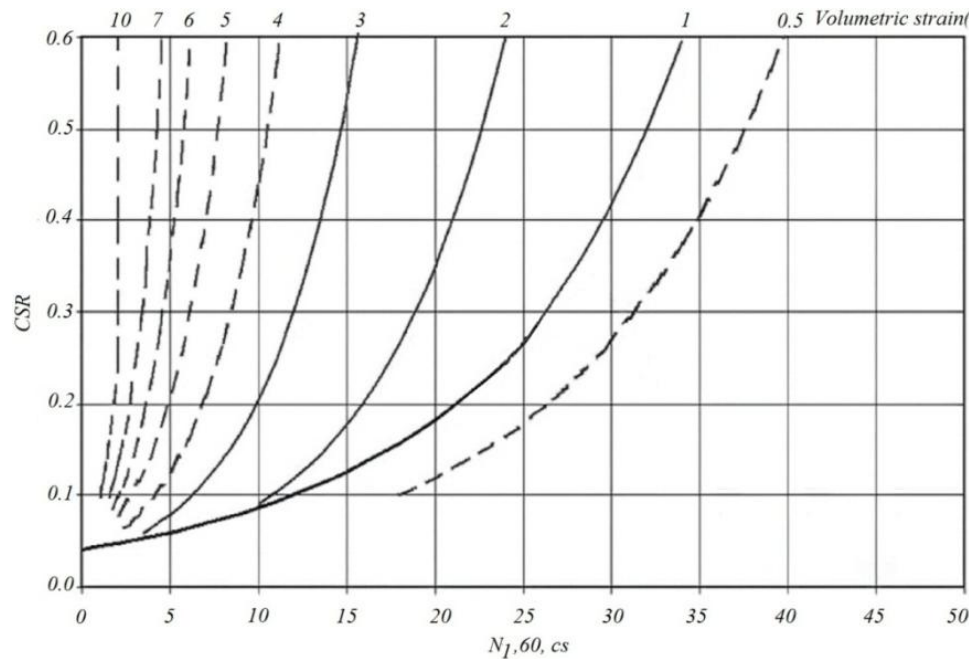


Fig. 3. The volumetric strain estimation based on the studies by Wu [29].

4. Methods for assessing the surface manifestation of liquefaction

The destructive effects of liquefaction on the ground and adjacent structures are of special importance. When the FS against liquefaction is less than one, the target soil layer will liquefy under the design earthquake conditions. However, it is necessary to mention that a low FS value in a soil layer does not necessarily mean the occurrence of overall failure at the ground surface. This is because a horizontal layer may liquefy during an earthquake without causing noticeable changes at the surface. Common analytical methods for determining the surface manifestation of liquefaction are presented as follows.

4.1. Iwasaki et al. method

Iwasaki et al. [30] expressed the severity of liquefaction occurrence at a site by introducing a parameter called the liquefaction potential index (P_L) as shown in Eq. 1:

$$P_L = \int_0^{20} F_L \cdot W(z) \cdot dz$$

(1)

where F_L is obtained based on the FS, as shown in Eq. 2:

$$F_L = \begin{cases} 1 - FS & FS \leq 1 \\ 0 & FS > 1 \end{cases}$$

(2)

$W(z)$ is a function of depth and is found by Eq. 3:

$$W(z) = \begin{cases} 10 - 0.5z & z \leq 20 \\ 0 & z > 20 \end{cases}$$

(3)

where z is the depth of the desired layer (m). Eq. 1 presents P_L values between 0 and 100. Based on this, Iwasaki et al. [30] provided Table 1 to relate P_L with the hazard level of each area.

Table 1. Relationship between P_L and hazard level.

Liquefaction hazard and required measures	P_L
Very low-detailed research, studies, and measurements are not required.	$P_L = 0$
Low-detailed investigations are required for important structures.	$0 < P_L \leq 5$
High-detailed investigations and further research are required for structures.	$5 < P_L \leq 15$
Very high-research, detailed investigations, and executive measures are required.	$15 < P_L$

4.2. Ishihara method

Ishihara [31], based on the idea that the occurrence of liquefaction in the underlying soil layers alone does not necessarily cause damage to structures located on or near the ground surface, proposed a method to determine the surface manifestation of liquefaction. According to empirical observations from previous earthquakes, Ishihara [31] suggested that if the ratio of the thickness of the non-liquefied surface layer (H_1) to the thickness of the underlying liquefied layer (H_2) is greater than certain specified values, significant surface manifestation of liquefaction will not occur. This threshold value is a function of the peak ground acceleration (PGA), as seen in Fig. 4.

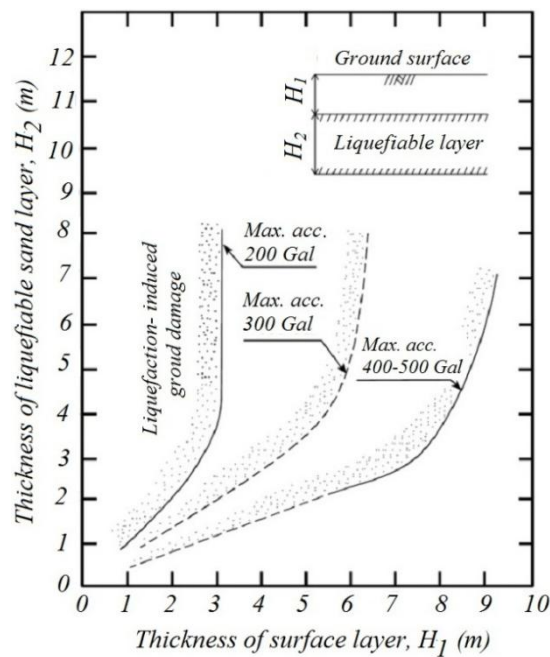


Fig. 4. Proposed boundary curves for determining the surface manifestation of liquefaction based on the studies of Ishihara [31].

5. Site and borehole characteristics

The geotechnical borehole used in this study is located in the southeastern region of Tehran, Iran, and was drilled to a depth of 20 m below the ground surface. Along with drilling the borehole, SPT, CPT, and Downhole tests were also conducted. The profile of the geotechnical borehole is shown in Fig. 5. The geotechnical properties of the layers are presented in Table 2, where γ , FC , D_{50} , D_r , $(N_1)_{60}$, V_s , q_c , and Rf are representative of the soil unit weight, percentage of fine particles, mean particle diameter, relative density, corrected SPT number, shear wave velocity, cone tip resistance, and friction ratio in CPT, respectively.

The geotechnical properties and field-test parameters presented in Table 2 were obtained directly from the in-situ and laboratory geotechnical investigations conducted at the study site. The dataset includes the results of SPT, CPT, and downhole testing, together with the relevant soil properties determined from laboratory testing. These measured and experimentally determined parameters were directly used as input data in the corresponding empirical liquefaction assessment methods.

For the liquefaction assessments presented in this study, a design earthquake scenario with moment magnitude $M_w = 7.5$ and peak ground acceleration $PGA = 0.35$ g was adopted. The magnitude $M_w = 7.5$ is used as the reference earthquake for the Tokimatsu and Seed [27] settlement estimation method, as well as for the magnitude scaling factor adjustments in the triggering methods. The $PGA = 0.35$ g corresponds to the design basis earthquake for the study area according to the Iranian seismic code and is used in the Ishihara [31] surface manifestation assessment. These parameters are consistently applied across all methods presented in Sections 6 through 8.

Depth (m)	Symbol	G.W. Level (m)	Description
1			Fill materials
2			SM
3			
4			
5			Silty sand with gravel
6			
7			ML
8			Silt with sand & gravel
9			SM
10			
11			
12			
13			ML
14			
15			
16			
17			
18			
19			
20			Silt with sand & gravel

Fig. 5. Geotechnical borehole.

Table 2. Geotechnical properties and field test results in the borehole.

Depth (m)	γ (kN/m ³)	FC (%)	D_{50} (mm)	D_r (%)	$(N_1)_{60}$	V_s (m/s)	q_c (MPa)	Rf (%)
2	17.2	41	0.13	42	16	193	4.8	1.90
4	17.3	43	0.15	45	17	200	5.6	3.30
6	17.4	48	0.08	50	20	220	7.8	3.60
8	18	51	0.8	50	18	208	6.2	3.00
10	17.6	15	1.5	38	12	180	5.4	2.50
12	18	20	1.4	48	19	202	7.2	2.80
14	17.8	60	0.07	51	20	215	7.9	3.20
16	17.1	90	0.003	53	21	217	7.1	3.30
18	17.4	85	0.003	55	23	223	9.2	3.10
20	18.8	78	0.02	57	24	233	10	3.20

6. Results of liquefaction potential assessment

Using the methods introduced in Section 2, the liquefaction potential in the borehole was evaluated. Fig. 6 illustrates the variations of CSR, CRR, and FS at different depths. It should be noted that the general nature of the Andrus and Stokoe [24] method differs from the other approaches, as previously stated, being based on shear wave velocity measurements. As shown in Fig. 6, within the first 8 m, due to the corrected shear wave velocity being greater than the threshold shear wave velocity, liquefaction will not occur. Since the velocities are smaller than this threshold, the FS cannot be calculated under these conditions, and only “non-occurrence of liquefaction” is reported. In fact, this situation highlights a weakness of the Andrus and Stokoe [24] method, because even when liquefaction is assumed not to occur, it is not possible to provide an FS at these depths. According to the previous investigations [30], even in the absence of liquefaction, if FS against liquefaction is less than 2, strength reduction exists and should be considered-something that cannot be evaluated using the Andrus and Stokoe [24] method.

The values obtained from different methods for calculating CRR are shown in Fig. 6a. The CRR values from Andrus and Stokoe [24] and Moss et al. [25] are smaller than the others. At depths of 8 to 14 m, the results of almost all methods are in good agreement with each other. Iwasaki et al. [23] method, particularly at greater depths, shows a significant difference compared to other methods. The reason for this difference is the presence of fine-grained particles at those depths (Table 2). By examining the CRR in Iwasaki et al. [23] method, the high importance of the percentage of fine particles and the mean particle size becomes evident, which justifies this discrepancy.

Fig. 6b shows the CSR results obtained from different methods of evaluating liquefaction potential. As seen, the methods of Boulanger and Idriss [26] and Andrus and Stokoe [24] display identical results due to similar relationships in determining CSR. Iwasaki et al. [23] method provides larger values for several reasons, such as not using the magnitude scaling factor and differences in the stress reduction coefficient relationships. Considering these points, it should be noted that the Iwasaki et al. [23] method is more conservative than other methods in measuring CSR. However, this conservatism applies only in this part. Considering the high CRR values, FS at some depths is much larger than in other methods. Another important point is that the CSR values obtained from Moss et al. [25] are smaller than those of other methods. The reason is the use of much more extensive field data (2153 case studies) compared to other approaches (a few hundred case studies) in determining the stress reduction coefficient relationship.

Fig. 6c compares the FS obtained from the mentioned methods. As evident, in terms of predicting liquefiable and non-liquefiable layers (regardless of the numerical value of the FS), not even two methods provide exactly the same results. This once again highlights the importance of more detailed and comprehensive evaluation of liquefaction potential and the inadequacy of existing engineering approaches.

The Iwasaki et al. [23] method indicates relatively similar results to other methods at depths of 7 to 13 m, but at other depths its results are generally different. The reason for this is the significant effect of considering the mean grain size in this method, which is ignored in other methods. This has caused the FS values in soils containing fine particles to increase sharply, which runs counter to conservatism. In fact, when the soil profile contains a large amount of fines, using this method is not very safe, as it may overestimate the FS compared to the actual value.

Since the Moss et al. [25] method results in smaller CSR values, it provides larger FS, such that at many depths, the FS is greater than that of other methods. The Boulanger and Idriss [26] method, although its results at many depths are reasonably similar to average values, still shows fundamental differences at some depths in terms of liquefaction occurrence or non-occurrence compared to the average results. At depths of 5 to 13 m, all methods, regardless of the numerical value of the FS, perform equally in predicting liquefiable and non-liquefiable layers. Another important point is that the average results of all methods from depths of 15 to 18 m are influenced by the very high FS values of the Iwasaki et al. [23] method, reporting FS greater than 1, while the results of other methods differ. Therefore, it can be stated that simply averaging the results of the methods and presenting a single number as the FS is not an accurate approach and may lead to misleading conclusions.

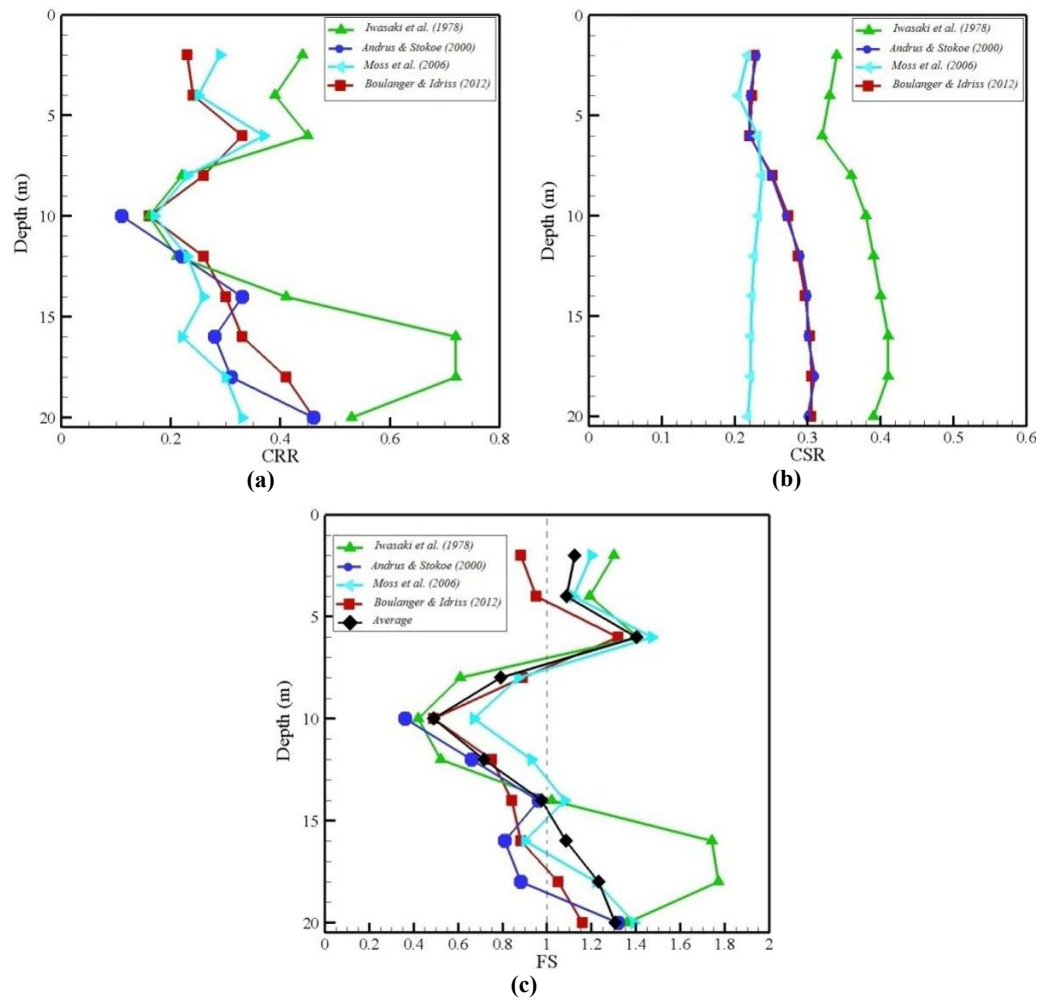


Fig. 6. Liquefaction potential assessment using different methods: (a) CRR; (b) CSR; and (c) FS.

7. Results of post-liquefaction settlement

In this section, the settlement caused by liquefaction (obtained from different methods of FS evaluation) is presented and compared using the three common approaches introduced in the previous sections.

7.1. Tokimatsu and Seed method

In order to estimate the post-liquefaction strain for an earthquake with a magnitude of 7.5, Tokimatsu and Seed [27] proposed a relationship between the SPT number and CSR. Therefore, the results of the methods by Andrus and Stokoe [24] and Moss et al. [25] cannot be evaluated using this approach, since the mentioned methods are respectively based on shear wave velocity and CPT. Fig. 1 has been used in this study to estimate the volumetric strain of the liquefied layer, and the results are presented in Table 3. It can be observed that the settlement values obtained from the two methods are approximately close to each other. However, the amount of settlement in the Iwasaki et al. [23] method is slightly greater than that of Boulanger and Idriss [26].

Table 3. The amount of soil settlement obtained from different methods based on the Tokimatsu and Seed [27] method.

Depth (m)	Layer thickness (m)	$(N_1)_{60}$	Iwasaki et al. [23]			Boulanger and Idriss [26]		
			CSR	Volumetric strain (%)	Settlement (cm)	CSR	Volumetric strain (%)	Settlement (cm)
2	2	16	0.34	1.8	3.6	0.23	1.6	3.2
4	2	17	0.33	1.6	3.2	0.22	1.5	3
6	2	21	0.32	1.3	2.6	0.22	1	2
8	2	18	0.36	1.5	3	0.25	1.5	3
10	2	12	0.38	1.9	3.8	0.27	2.1	4.2
12	2	19	0.39	1.6	3.2	0.29	1.4	2.8
14	2	20	0.40	1.4	2.8	0.30	1.5	3
16	2	21	0.41	1.3	2.6	0.30	1.4	2.8
18	2	23	0.40	1.2	2.4	0.30	1.2	2.4
20	2	24	0.38	1.2	2.4	0.30	1.1	2.2
Total settlement of the soil (cm)					29.6			28.6

7.2. Ishihara and Yoshimine method

In this section, post-liquefaction settlement of the soil layer is calculated using the Ishihara and Yoshimine [28] method. For this purpose, Fig. 2 was used to calculate settlement, and its results are presented in Table 4. Fig. 2 shows the curves estimating volumetric strain as a function of the normalized SPT number (N_1), normalized cone tip resistance (q_{c1}) and the FS against liquefaction (instead of CSR). It is observed that the settlement values from the methods of Moss et al. [25] and Boulanger and Idriss [26] are approximately close to each other. Since these two methods use the results of two different field tests to evaluate liquefaction, the similarity in settlement values calculated from these methods is significant. Furthermore, the Iwasaki et al. [23] method indicates quite different results from other methods and even from its own results calculated by Tokimatsu and Seed [27]. The reason for this is the application of the FS in the Ishihara and Yoshimine [28] method. It is worth mentioning that the Andrus and Stokoe [24] method cannot be evaluated using this approach due to the use of shear wave velocity.

Table 4. The amount of soil settlement obtained from different methods based on the Ishihara and Yoshimine [28] method.

Depth (m)	Layer thickness (m)	$(N_1)_{60}$	q_{c1} (kg/cm ²)	Iwasaki et al. [23]			Boulanger and Idriss [26]			Moss et al. [25]		
				FS	Volumetric strain (%)	Settlement (cm)	FS	Volumetric strain (%)	Settlement (cm)	FS	Volumetric strain (%)	Settlement (cm)
2	2	16	49	1.30	0.6	1.2	0.88	1.5	3	1.20	0.55	1.1
4	2	17	57	1.19	0.6	1.2	0.95	1.3	2.6	1.12	0.65	1.3
6	2	21	80	1.41	0.4	0.8	1.32	0.6	1.2	1.47	0.2	0.4
8	2	18	63	0.61	2.4	4.8	0.89	1.5	3	0.87	3.5	7
10	2	12	55	0.42	3.2	6.4	0.49	3.3	6.6	0.67	3.8	7.6
12	2	19	73	0.52	2.1	4.2	0.75	1.8	3.6	0.93	2.0	4.0
14	2	20	81	1.02	1.3	2.6	0.84	1.5	3	1.08	0.8	1.6
16	2	21	72	1.74	0.3	0.6	0.88	1.4	2.8	0.90	2.9	5.8
18	2	23	94	1.77	0.3	0.6	1.05	0.7	1.4	1.22	0.45	0.9
20	2	24	102	1.36	0.5	1	1.16	0.6	1.2	1.39	0.4	0.8
Total settlement of the soil (cm)						23.4	28.4			30.5		

7.3. Wu method

In this section, the settlement of the soil layer is once again calculated using the Wu [29] method. For this purpose, Fig. 3 has been used to calculate the settlement, and its results are presented in Table 5. As seen, the settlement values resulting from the two methods are approximately close to each other. The settlement value in the Iwasaki et al. [23] method is slightly higher than that in the Boulanger and Idriss [26] method. This result is similar to the Tokimatsu and Seed [27] method and opposite to the Ishihara and Yoshimine [28] method. Another point is that the Wu [29] method calculates settlement based on the modified SPT number for clean sand. Therefore, the Andrus and Stokoe [24] and Moss et al. [25] methods cannot be examined using this approach because these methods are based on shear wave velocity and CPT, respectively.

Table 5. The amount of soil settlement obtained from different methods based on the Wu [29] method.

Depth (m)	Layer thickness (m)	$(N_1)_{60cs}$	Iwasaki et al. [23]			Boulanger and Idriss [26]		
			CSR	Volumetric strain (%)	Settlement (cm)	CSR	Volumetric strain (%)	Settlement (cm)
2	2	21	0.34	2	4	0.23	1.6	3.2
4	2	22	0.33	1.8	3.6	0.22	1.4	2.8
6	2	26	0.32	1.2	2.4	0.22	0.7	1.4
8	2	23	0.36	1.6	3.2	0.25	1.2	2.4
10	2	15	0.38	2.8	5.6	0.27	2.6	5.2
12	2	23	0.39	1.6	3.2	0.29	1.5	3
14	2	25	0.40	1.5	3	0.30	1.2	2.4
16	2	26	0.41	1.4	2.8	0.30	1.1	2.2
18	2	28	0.40	1.1	2.2	0.30	0.8	1.6
20	2	29	0.38	1	2	0.30	0.8	1.6
Total settlement of the soil (cm)					32	27.8		

7.4. Comparison of the results of post-liquefaction settlement

To calculate the post-liquefaction settlement in this study, three methods were used: Tokimatsu and Seed [27], Ishihara and

Yoshimine [28], and Wu [29]. Examination of the results from these three methods illustrates that the calculated liquefaction-induced settlements are in reasonably good agreement with each other. As evident, despite the overall consistency, the amount of settlement differs slightly among the methods, and if the data are to be applied to specific engineering structures such as towers, nuclear reactors, etc., further scrutiny of the results is required. For example, the post-liquefaction settlement calculated using the Iwasaki et al. [23] method differs by up to 8.6 cm across various approaches, which is a considerable amount. Likewise, the post-liquefaction settlement calculated using the Iwasaki et al. [23] and Moss et al. [25] methods compared with Ishihara and Yoshimine [28] displays a difference of 7.1 cm, which again is a significant discrepancy.

The differences among the estimated post-liquefaction settlement can be attributed to differences in the input parameters, empirical formulations, calibration datasets, and underlying assumptions of the three methods. In addition, each method has been developed using specific experimental or field conditions and therefore may exhibit different sensitivity to soil density, cyclic stress level, and liquefaction severity. Consequently, the differences observed in the present study should not necessarily be interpreted as an indication that one method is universally superior to the others.

8. Results of surface manifestation

In this section, the surface manifestation is evaluated using the calculated FS obtained through different methods, with the aid of the approaches proposed by Iwasaki et al. [30] and Ishihara [31]. These evaluations are then compared with each other to provide a more comprehensive understanding of ground conditions after the occurrence of liquefaction.

8.1. Iwasaki et al. method

According to Eqs. 1 to 3 and Table 1, using the calculated FS values from each method, the P_L has been computed and presented in Table 6. As observed, the Iwasaki et al. [30] method evaluates surface manifestation qualitatively. However, it has an advantage over the Ishihara [31] method by considering the FS values. Based on the results presented in Table 6, the probability of failure according to the Iwasaki et al. [23] method is higher than that of other methods. The probability of failure using the Moss et al. [25] method is much lower than the other methods, though further detailed investigations are still required. It should be mentioned that since the database used to establish the Moss et al. [25] method was larger than that of the other methods, a higher FS has been reported. It is also worth noting that this method is based on CPT data, which provides continuous subsurface profiling and can offer advantages in identifying spatial variations in soil resistance compared with discrete SPT measurements.

Table 6. Evaluation of surface manifestation obtained from different methods based on the Iwasaki et al. [30] method.

Depth (m)	Iwasaki et al. [23]				Andrus and Stokoe [24]				Moss et al. [25]				Boulanger and Idriss [26]			
	FS	F_L	$W(z)$	$\frac{W(z)}{\times F_L}$	FS	F_L	$W(z)$	$\frac{W(z)}{\times F_L}$	FS	F_L	$W(z)$	$\frac{W(z)}{\times F_L}$	FS	F_L	$W(z)$	$\frac{W(z)}{\times F_L}$
2	1.3	0	9	0	-	-	-	-	1.20	0	9	0	0.88	0.12	9	1.08
4	1.18	0	8	0	-	-	-	-	1.12	0	8	0	0.95	0.05	8	0.4
6	1.41	0	7	0	-	-	-	-	1.47	0	7	0	1.32	0	7	0
8	0.61	0.39	6	2.34	-	-	-	-	0.87	0.13	6	0.78	0.89	0.11	6	0.66
10	0.42	0.58	5	2.9	0.36	0.64	5	3.2	0.67	0.33	5	1.65	0.49	0.51	5	2.55
12	0.52	0.48	4	1.92	0.66	0.34	4	1.36	0.93	0.07	4	0.28	0.75	0.25	4	1
14	1.02	0	3	0	0.96	0.04	3	0.12	1.08	0	3	0	0.84	0.16	3	0.48
16	1.74	0	2	0	0.81	0.19	2	0.38	0.90	0.10	2	0.2	0.88	0.12	2	0.24
18	1.77	0	1	0	0.88	0.12	1	0.12	1.22	0	1	0	1.05	0	1	0
20	1.36	0	0	0	1.32	0	0	0	1.39	0	0	0	1.16	0	0	0
P_L	7.16				5.18				2.91				6.41			
Description	High liquefaction risk (severe ground surface damage)				High liquefaction risk (severe ground surface damage)				Low liquefaction risk (requires detailed investigations)				High liquefaction risk (severe ground surface damage)			

8.2. Ishihara method

Upon examining the results obtained from various methods (Fig. 6c), it is observed that the thickness of liquefiable and non-liquefiable layers differs across all liquefaction assessment methods. The surface manifestation was evaluated according to Ishihara [31] method (Fig. 4), the values from Fig. 6c, and considering the $PGA = 0.35$ g (350 Gal). Table 7 presents the results of this evaluation. According to Ishihara [31], the Andrus and Stokoe [24] method shows no surface manifestation. In contrast, surface manifestation is observed in the Moss et al. [25] method. This issue does not align with the results of Iwasaki et al. [30], further emphasizing the importance of adopting a comprehensive approach to assess liquefaction effects in a region. It should also be noted that the Ishihara [31] method qualitatively examines surface manifestation, making direct comparisons between methods infeasible.

For instance, surface manifestation due to liquefaction is evident in the methods of Iwasaki et al. [23], Moss et al. [25], and Boulanger and Idriss [26], but it remains unclear which method indicates the greatest intensity of this phenomenon.

Table 7. Evaluation of surface manifestation obtained from different methods based on the Ishihara [31] method.

Method	Surface manifestation
Iwasaki et al. [23]	Yes
Andrus and Stokoe [24]	No
Moss et al. [25]	Yes
Boulanger and Idriss [26]	Yes

8.3. Comparison of surface manifestation results

In determining the extent of surface manifestation due to liquefaction, the methods of Iwasaki et al. [30] and Ishihara [31] were employed. As stated, Iwasaki et al. [30] method, similar to that of Ishihara [31] method, qualitatively evaluates surface manifestation; however, considering the inclusion of FS values, it has an advantage over Ishihara [31] method. Based on the approach proposed by Iwasaki et al. [30], surface manifestation resulting from liquefaction is observed in all methods except that of Moss et al. [25]. According to Ishihara [31] method, surface manifestation exists in all methods except Andrus and Stokoe [24]. This situation highlights the importance of employing multiple methods simultaneously to assess such a critical issue. According to the results of Iwasaki et al. [30], the damage estimated by Iwasaki et al. [23] is greater than that of other methods (larger P_L). However, Andrus and Stokoe [24], among the methods that assess the surface manifestation, has the least damage (smaller P_L), which is consistent with Ishihara [31] method.

9. Providing a strategy for evaluating liquefaction potential

Based on the results obtained from multiple evaluations conducted in this study, and the considerable discrepancies observed among different methods in determining liquefaction potential, post-liquefaction settlement, and surface manifestations, it is evident that the reliance on a single criterion or a single field test cannot provide an accurate assessment of soil behavior under seismic excitation. The findings indicate that methods based on SPT, CPT, and V_s each predict only a portion of soil behavior, and none alone is capable of presenting a comprehensive picture of the soil response. Furthermore, the mechanism of liquefaction is not a one-dimensional phenomenon, and merely the FS does not allow for reliable judgment regarding the severity of damage, post-liquefaction settlement, or the likelihood of failure propagation to the ground surface. Accordingly, in this section, a multi-stage framework is proposed with the aim of reducing the risk of relying on a single empirical procedure and providing a more comprehensive basis for engineering decision-making. This framework simultaneously considers the three principal components of liquefaction (potential occurrence, settlement, and surface manifestation) and utilizes the results of several different methods for final decision-making. Therefore, the framework consists of four essential stages as follows:

1. Assessment of liquefaction potential based on a multi-criteria approach
2. Calculation of post-liquefaction settlement
3. Evaluation of surface manifestation
4. Integration of results and final hazard classification

The novelty of the proposed framework does not lie in developing a new empirical equation for liquefaction triggering or post-liquefaction deformation. Rather, its main contribution is the systematic integration of three complementary aspects of liquefaction hazard, including triggering potential, post-liquefaction settlement, and surface manifestation, within a sequential engineering decision-making procedure. By combining different SPT-, CPT-, and V_s -based assessments with deformation and surface manifestation criteria, the framework provides an integrated interpretation of liquefaction consequences rather than relying solely on a single factor of safety.

9.1. Assessment of liquefaction potential based on a multi-criteria approach

Given the analyses performed, the necessity of using several different methods is felt. The analyses of this study showed that differences between methods arise from variations in the type of field test (SPT, CPT, and V_s), differences in theoretical assumptions in calculating CSR and CRR, differing sensitivities of methods to fines content, relative density, and stress history, the empirical nature of many relationships, and limitations of primary databases. Therefore, it is recommended that the initial assessment of liquefaction potential be performed using three different field-test-based methods (SPT, CPT, and V_s). Although these methods are not fully independent, as they are all influenced by common soil parameters such as density and effective stress, their combination can help reduce the influence of method-specific biases and uncertainties. This overlap of information compensates for weaknesses or errors of any one method by the other two.

It is also possible to use a separate decision-making system for each soil layer. For each depth layer, decisions can be made as follows: If two methods report $FS < 1$, the layer is considered liquefiable. The 'two out of three' criterion is adopted as a practical screening rule rather than a probabilistic threshold. It provides a compromise between the conservative interpretation of a single method (which may produce a false positive due to method-specific biases) and the potentially misleading requirement that all

methods must simultaneously indicate liquefaction (which may produce a false negative due to differing method sensitivities). This approach is analogous to the concept of 'consensus-based' decision-making in multi-model ensemble analyses, where the majority vote is considered more reliable than any single model output. If only one method reports $FS < 1$, the layer is considered suspect and requires further analysis. Finally, if all methods report $FS > 1$, the layer is stable. This approach is similar to multi-model probabilistic decision-making methods in seismic analyses and reduces uncertainty.

9.2. Calculation of post-liquefaction settlement

The research results indicate that the FS alone is insufficient for hazard assessment, as two layers may both have FS values near 1, yet exhibit vastly different settlement; one layer might liquefy but cause negligible settlement due to adequate density, while certain structures (e.g., pipelines, sensitive foundations, tanks) are more sensitive to settlement than to liquefaction itself.

To calculate settlement, some methods are applied based on diverse parameters such as cyclic stress, SPT number, FS, relative density, or large-scale laboratory data. The arithmetic mean of the settlement estimates obtained from the three methods is used as a practical engineering indicator of probable settlement (S), rather than as a statistically rigorous estimate of the true settlement. Since the three empirical methods are based on different assumptions, input parameters, and calibration datasets, their estimates cannot necessarily be considered statistically independent. The averaged value is therefore intended to provide a practical basis for the subsequent screening and classification within the proposed framework. Based on the settlement criteria and guidance provided in reputable foundation design codes and standards, including Eurocode 7 [32], BS 8004:2015+A1 [33], JGS 4001-2004 [34], and National Building Regulations of Iran, Topic 7 [35], settlement thresholds of 2 cm and 5 cm are adopted in this study as practical screening limits, leading to the following decision rule:

If $S < 2$ cm, liquefaction effects are negligible but should be added to total structural settlement and considered. If $2 \text{ cm} < S < 5$ cm, further review and numerical analyses for specific structures are required. If $S > 5$ cm, settlement is significant, necessitating soil improvement. Moreover, when the settlement estimates differ by more than 30%, the discrepancy should be regarded as an indication of increased uncertainty. The 30% discrepancy threshold is adopted based on common engineering practice and is intended to flag cases where the settlement estimates from different methods are sufficiently divergent to warrant additional scrutiny. In such cases, the CPT-based estimate or the estimate associated with the lower factor of safety may be given greater consideration, provided that the quality and applicability of the corresponding field data and empirical correlation are confirmed. These values are adopted as practical screening thresholds for the present framework and should not be interpreted as universal settlement limits for all types of structures. The acceptable level of liquefaction-induced settlement depends on structural characteristics, foundation type, serviceability requirements, and site-specific conditions. Therefore, the proposed limits are intended to identify cases requiring further engineering evaluation rather than to replace structure-specific settlement criteria.

It is worth noting that the approach used for settlement estimation (arithmetic averaging of three different estimates) differs from the approach used for FS evaluation (majority-rule decision-making). This distinction is intentional and reflects the different nature of the two assessments. For FS, the outcome is binary (liquefaction vs. non-liquefaction), and a single method with an FS slightly below 1 may be subject to method-specific biases; therefore, the majority-rule approach (two out of three) is more conservative and robust. For settlement, however, the outcome is a continuous variable (displacement in centimeters) and averaging multiple different estimates has been shown to reduce the influence of method-specific overestimation or underestimation, providing a more balanced engineering estimate.

9.3. Evaluation of surface manifestation of liquefaction

Even if $FS < 1$, liquefaction may occur at depth without producing any surface effects. The opposite may also happen, meaning FS slightly greater than 1, but due to the thinness of the surface layer, the ground may experience sand boiling and lateral spreading. Therefore, this stage complements the previous two stages and is critical for the safety of surface structures. Using reliable methods such as those presented in this study, surface manifestation should be evaluated, and decisions should be made according to the following rule:

1. If both methods indicate surface manifestation, a high likelihood of surface manifestation is inferred.
2. If only one method reports surface manifestation, the layer is suspicious and numerical analysis (FLAC, PLAXIS) or supplementary testing must be performed.
3. If both methods indicate no surface manifestation, surface manifestation is considered unlikely based on the adopted criteria, although site-specific uncertainties should still be considered.

9.4. Integration of results and final hazard classification

For the final decision, three main indicators should be integrated. In fact, the potential for liquefaction (FS), the severity of settlement (S), and the probability of surface manifestation (P_L and H_1/H_2) should be taken into account. Based on this, four levels of risk are defined, which are presented in Table 8. For the final classification, each of the three indicators is first evaluated independently: liquefaction triggering potential based on FS, liquefaction-induced settlement, and surface manifestation. The final hazard group is then governed by the most critical classification among these indicators. This conservative approach ensures that a potentially severe consequence is not masked by less critical values of the other indicators. The FS thresholds presented in Table 8 (0.8, 1.0, and 1.5) are adopted based on common engineering practice and recommendations in the literature. A threshold of FS =

1.0 represents the conventional boundary between liquefaction and non-liquefaction in deterministic analyses [20, 23, 26]. The upper threshold of $FS = 1.5$ is consistent with the recommendation by several codes and guidelines that require $FS \geq 1.2$ -1.5 for critical structures [4, 32-35]. The lower threshold of $FS = 0.8$ is adopted to identify layers where liquefaction is highly probable (i.e., seismic demand exceeds resistance by more than 25%) [4]. It should be emphasized that these thresholds are intended as practical screening criteria based on engineering judgment and current practice, and they may require adjustment for specific projects or regulatory requirements.

Table 8. Final classification of liquefaction hazard.

Criteria used for final liquefaction hazard classification	Group number			
	1	2	3	4
FS with multi-criteria approach	<1.5	1-1.5	0.8-1	0.8<
Settlement with multi-criteria approach (cm)	2<	2-5	5-10	<10
P_L	0	0-5	5-15	<15
Ratio of (H_1/H_2)	High	Medium	Low	Negligible
Risk level	Low	Medium	High	very high
Necessary actions	No specific action	Detailed investigation	Ground improvement	Definitive improvement/ design Rejection

The proposed approach provides several practical advantages, including the simultaneous consideration of SPT, CPT, and V_s information; integrated assessment of liquefaction triggering, post-liquefaction settlement and surface manifestation; and applicability to conventional engineering screening without requiring complex numerical analyses. The framework may also provide a basis for future extensions using probabilistic or data-driven approaches. Further validation using multiple case histories and geological settings is required before its use as a generalized assessment procedure or guideline framework.

9.5. Limitations and future applications

Despite the practical advantages of the proposed framework, several limitations should be acknowledged. First, the framework was demonstrated using a geotechnical borehole and therefore does not constitute a comprehensive validation under different geological and seismic conditions. Second, the input field measurements themselves are subject to inherent uncertainties. SPT blow counts can vary due to operator effects, energy transfer efficiency, and soil disturbance; CPT tip resistance and friction ratio are influenced by penetrometer type, rate of penetration, and soil fabric interpretation; and shear-wave velocity measurements obtained from downhole testing are affected by source-receiver coupling, signal processing, and layering assumptions. Third, the proposed settlement thresholds and decision rules are intended as practical screening criteria and should not replace structure-specific geotechnical and seismic analyses. Finally, liquefaction-related consequences such as lateral spreading, bearing capacity loss, and structure-specific differential settlement are not explicitly incorporated into the present framework.

10. Conclusions

The evaluation of liquefaction potential is considered as one of the most challenging aspects of geotechnical earthquake studies. The findings of this research indicate that the reliance on a single method of liquefaction potential assessment based on SPT, CPT, or V_s cannot provide a realistic and comprehensive picture of soil behavior during earthquakes. Significant discrepancies among methods in calculating CSR, CRR, and FS indicate that soil response under seismic loading is highly multifactorial and dependent on the specific characteristics of each layer, and single-criterion approaches cannot adequately meet engineering design requirements. Moreover, post-liquefaction settlement and surface manifestation are critical factors that unfortunately do not have a prominent role in current evaluation practices. In fact, situations may occur where the FS is greater than one, yet the structure experiences considerable settlement problems.

Data analysis further revealed that some methods behave conservatively at certain depths and non-conservatively at others, showing varying sensitivity to fines content, relative density, stress history, and analytical parameters. This highlights the potential value of adopting an integrated, multi-criteria framework as a practical basis for liquefaction assessment. The main findings of this study can be summarized as follows:

1. Performance differences among methods in determining liquefaction potential are striking. Methods based on SPT, CPT, and V_s did not provide consistent results, and discrepancies in FS were repeatedly observed. None of the methods even produced similar patterns in identifying liquefiable layers. This indicates that simple averaging of FS values obtained from different methods, without considering other geotechnical conditions, can be misleading.
2. Although the three methods of Tokimatsu and Seed [27], Ishihara and Yoshimine [28], and Wu [29] provided generally consistent and acceptable results in estimating post-liquefaction settlement, in some cases differences of up to 8 cm were observed. Such discrepancies are critical and unacceptable for sensitive structures. Moreover, the shear wave velocity-based method cannot be directly applied in this part, revealing another weakness of this method.
3. Differences in evaluating surface manifestation of liquefaction were observed. The Iwasaki et al. [30] method indicated high surface manifestation for most cases, except for the Moss et al. [25] method. Ishihara [31] confirmed that the Andrus and Stokoe [24] method showed no surface manifestation, while other methods considered it likely. This significant

discrepancy again demonstrated that surface manifestation cannot be determined by a single method. It is also important to employ an approach that provides not only qualitative but also quantitative estimates of surface manifestation due to liquefaction.

4. The findings demonstrate that liquefaction should be assessed simultaneously from three perspectives: triggering potential, settlement, and surface manifestation. The proposed four-step multi-criteria framework, integrating data from three field tests, three settlement estimation methods, and two surface manifestation evaluation methods, provides a more comprehensive engineering-oriented screening assessment than any single method alone.
5. The proposed framework demonstrates potential as a practical screening tool for liquefaction assessment in similar geotechnical conditions. It is expandable and can be used without the need for complex numerical analyses in preliminary engineering studies. However, it should be emphasized that the framework has been developed and demonstrated using a single borehole. Further validation using independent case histories, multiple boreholes, and sites with different geological and seismic characteristics is essential before the framework can be generalized as a standard assessment procedure. The approach represents a preliminary step toward reducing uncertainty in liquefaction evaluation, but its applicability to other regions and conditions requires additional verification.

Statements & declarations

Author contributions

Sadegh Rezaei: Conceptualization, Methodology, Validation, Formal analysis, Writing - Original Draft, Supervision.

Ali Hasanzadeh: Conceptualization, Methodology, Formal analysis, Writing - Review & Editing.

Majid Moradi: Methodology, Validation, Formal analysis, Writing - Review & Editing.

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Data availability

The data presented in this study are available on request from the corresponding author. The data are not publicly available due to data ownership and usage restrictions.

Declarations

The authors declare no conflict of interest.

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Hydrolytic Enzyme Effects on Total Coliforms in Surfactant-Rich Industrial Wastewater and Soil Extracts: A Control-Adjusted Assessment

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ABSTRACT

Detergent manufacturing wastewater contains surfactants, organic contaminants, and microbial populations that may challenge conventional treatment processes. This study evaluated the response of total coliforms to hydrolytic enzyme treatments in detergent-impacted industrial wastewater and aqueous soil extracts under controlled laboratory conditions. Cellulase, amylase, protease, an enzyme-supplemented detergent formulation, an enzyme-free detergent formulation, and untreated controls were assessed during a 7-day incubation period. Total coliform abundance was determined using the five-tube Most Probable Number (MPN) method, and treatment performance was evaluated using endpoint abundance and residual coliform counts. A substantial decline occurred in the untreated wastewater control, which exhibited an 88.5% reduction in coliform abundance, indicating that natural attenuation represented a major component of the overall response. Among wastewater treatments, cellulase (WW-CEL) and the enzyme-supplemented detergent formulation (WW-D+E) exhibited the lowest endpoint coliform abundances, with endpoint values at or near the analytical detection limit. Relative to the untreated control, WW-CEL was associated with an additional reduction of approximately 11.4% beyond the baseline decline observed under the experimental conditions. In contrast, the enzyme-free detergent treatment produced an endpoint response comparable to the untreated control. Most soil extract treatments exhibited baseline coliform abundances at or below the analytical detection limit, limiting interpretation of treatment-specific responses. These findings suggest that hydrolytic enzymes may contribute to incremental reductions in coliform abundance in surfactant-rich industrial wastewater. However, the overall response was strongly dominated by natural attenuation, and observed treatment effects were highly dependent on matrix characteristics and baseline microbial load.

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1. Introduction

The detergent manufacturing sector represents one of the largest industrial contributors to aquatic pollution, with global production exceeding 15 million tonnes annually and generating complex effluents that challenge conventional wastewater treatment systems. These wastewaters typically contain high concentrations of surfactants, particularly linear alkylbenzene sulfonates (LAS), residual organic matter, and enzymatic additives, forming chemically heterogeneous matrices that complicate microbial control and treatment stability [1]. In addition to chemical pollutants, such effluents frequently carry fecal indicator organisms such as total coliforms and *Escherichia coli*, which are widely used as regulatory benchmarks for assessing microbiological water quality and treatment performance [2].

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Recent evidence from advanced wastewater treatment and bioreactor systems indicates that modern industrial effluents are increasingly characterized by multi-contaminant stress environments, where surfactants, pharmaceuticals, and emerging organic compounds simultaneously reshape microbial community structure and process stability, often reducing the predictability of conventional treatment performance [3]. In parallel, studies on surfactant-acclimated microbial consortia have demonstrated that long-term exposure to linear alkylbenzene sulfonates induces functional adaptation, but also results in altered degradation kinetics and community-level tolerance thresholds [4, 5]. These observations suggest that wastewater systems exposed to complex industrial chemistries cannot be fully interpreted using simplified or single-stressor assumptions.

Conventional disinfection strategies, including chlorination, ultraviolet (UV) irradiation, and ozonation, remain widely implemented for pathogen control. However, their effectiveness is frequently compromised in surfactant-rich wastewater matrices. Elevated LAS concentrations reported in detergent manufacturing effluents have been reported to reduce disinfectant efficiency through competitive adsorption, protection of microbial aggregates, and interference with oxidative reactions [4, 6]. Moreover, conventional chemical disinfection may generate secondary pollutants such as halogenated byproducts, while UV-based systems exhibit reduced efficiency in turbid or high-solids environments and may fail against resistant microbial forms [5, 7, 8]. Collectively, these limitations have contributed to increasing interest in integrated or bioaugmented treatment strategies capable of addressing both chemical complexity and microbial persistence under industrial conditions.

The structural integrity of bacterial pathogens, particularly fecal coliforms, is governed by the hierarchical architecture of the cell envelope, comprising peptidoglycan networks and associated carbohydrate-rich structures that provide mechanical stability and environmental protection [9]. Enzymatic targeting of these structures offers a mechanistically distinct approach to microbial control. Hydrolytic enzymes such as cellulases, proteases, and amylases can degrade extracellular and structural polymeric components associated with microbial cells and biofilms, potentially weakening microbial protective barriers and enhancing susceptibility to external stressors [10-13]. In this context, enzyme-based biocatalytic approaches have been increasingly explored not only for pollutant transformation but also for targeted biological disruption in complex environmental systems, including wastewater and sludge matrices [3, 14].

Recent studies on enzymatic and biological treatment systems further support this direction, demonstrating that enzyme-assisted or bioaugmented processes can influence microbial viability and biofilm-associated tolerance mechanisms in engineered environments, particularly under conditions of high organic and particulate loading [15-19]. For example, hydrolytic enzymes have been reported to disrupt microbial biofilms and extracellular polymeric substances, thereby enhancing susceptibility to chemical or environmental stressors, although their performance remains strongly dependent on system composition and matrix structure [17-19]. Similarly, bioaugmentation strategies in wastewater and sludge systems have highlighted the importance of microbial adaptation and system-specific metabolic pathways in determining overall treatment efficiency [20, 21].

Despite these advances, most existing studies have focused on simplified laboratory systems or model substrates that do not fully capture the complexity of industrial effluents. In particular, enzyme performance in the presence of surfactants such as LAS, which themselves influence microbial physiology and interfacial interactions, remains insufficiently characterized. Additionally, matrix-dependent effects including adsorption to particulate matter, sludge-associated inhibition, and microbial community restructuring have been identified as key determinants of treatment variability in real wastewater systems [7, 8, 22].

However, quantitative evidence describing the response of coliform populations to hydrolytic enzymes under surfactant-rich industrial wastewater conditions, while simultaneously accounting for matrix-dependent effects and natural attenuation processes, remains limited. In this context, the present study investigates the response of total coliform populations to hydrolytic enzyme treatments in two environmentally relevant matrices: (I) detergent manufacturing wastewater characterized by high surfactant and organic loading, and (II) aqueous soil extract representing a lower-load environmental system. This design is motivated by evidence that contaminant behavior, microbial persistence, and treatment efficiency differ substantially between wastewater and solid-associated environmental matrices due to adsorption, bioavailability, and community-level constraints [7, 8, 21].

To our knowledge, few studies have simultaneously evaluated hydrolytic enzyme treatments in surfactant-rich industrial wastewater while explicitly accounting for natural attenuation processes and comparing responses across contrasting environmental matrices. This combination of enzyme treatment, matrix comparison, and control-adjusted interpretation constitutes the primary contribution of the present study. We hypothesize that hydrolytic enzymes may contribute to measurable reductions in coliform abundance in detergent-impacted wastewater; however, the magnitude and interpretability of these effects are expected to be strongly dependent on matrix composition, microbial load, and physicochemical constraints rather than enzymatic activity alone.

The objectives of this study are therefore: (1) to quantify changes in culturable total coliform abundance following exposure to cellulase, amylase, and protease; (2) to evaluate enzyme-associated responses relative to untreated controls using endpoint-based statistical analysis; (3) to assess matrix-dependent differences in wastewater and soil extract systems; and (4) to develop a conservative interpretative framework for enzyme-assisted treatment in surfactant-rich industrial effluents.

2. Materials and methods

2.1. Study site and sample collection

2.1.1. Industrial wastewater

Composite wastewater samples were collected from the effluent discharge point of Active Co. detergent manufacturing facility

(Eshtehard Industrial Zone, 35°42'N, 50°22'E, Alborz Province, Iran). The facility produces household and industrial cleaning products containing amylase, cellulase, and protease. The effluent comprises process rinse streams, equipment washdown water, and incidental spillage, all discharged through a common collection channel.

Triplicate composite samples were independently collected at three time points (0, 15, and 30 min) to capture temporal variability in discharge composition. Each 500 mL sample was collected in sterile borosilicate glass bottles and transported at 4 °C to the Environmental Engineering Laboratory, University of Tehran, within 3 h of collection, complying with APHA holding-time protocols [23].

2.1.2. Soil aqueous extract

Soil samples were collected at six spatially distributed points (0–30 cm depth) using a stainless-steel corer and combined into a single composite per replicate (triplicate sets collected at 0, 15, and 30 min). Aqueous extracts were prepared by mixing 100 g of air-dried soil (2 mm sieved) with 500 mL of sterile deionised water, shaking for 60 min at 150 rpm, and filtering through a Whatman No. 42 filter paper [21]. Extracts were processed identically to wastewater samples within 6 h of preparation.

2.2. Physicochemical characterisation of matrices

Baseline physicochemical parameters were measured at sample receipt (T_0) using calibrated instruments: pH and dissolved oxygen (DO) by portable multi-parameter meter (HQ40d, Hach); temperature by integrated thermistor; turbidity by turbidimeter (2100Q, Hach); COD in wastewater by closed reflux titrimetric method (APHA 5220-C); anionic surfactant LAS by MBAS colorimetric method (APHA 5540-C); electrical conductivity in soil extract by conductivity meter (HQ40d, Hach). These treatment-specific baseline values are therefore reported with the coliform enumeration results rather than in Table 1.

Table 1. Physicochemical characteristics at baseline (T_0).

Parameter	Industrial wastewater	Soil extract	Method
PH	7.2 ± 0.3	6.8 ± 0.4	PH meter (HQ40d)
DO (mg/L)	2.4 ± 0.6	5.1 ± 0.8	LDO probe (HQ40d)
Temperature (°C)	22.5 ± 1.2	20.8 ± 0.9	Integrated thermistor
Turbidity (NTU)	148 ± 22	38 ± 11	Turbidimeter (2100Q)
COD (mg/L)	1,240 ± 180	—	Closed reflux (APHA 5220-C)
LAS (mg/L)	48 ± 9	—	MBAS (APHA 5540-C)
Conductivity (µS/cm)	—	620 ± 45	Conductivity meter

• Values are mean ± SD (n = 3). LAS = linear alkylbenzene sulfonates. Treatment-specific baseline coliform concentrations were determined separately after treatment allocation and are reported with the coliform enumeration results rather than as physicochemical matrix characteristics.

Experimental rationale: The measured pH (7.2 wastewater, 6.8 soil extract) falls within the optimal range for the three enzyme preparations (pH 6.5–7.5 for amylase/cellulase; pH 7.0–8.5 for alkaline protease). However, wastewater DO (2.4 mg/L) was below the threshold for optimal aerobic metabolism (> 4 mg/L); thus, 150 rpm orbital shaking maintained adequate aeration throughout incubation.

2.3. Preparation of enzyme stock solutions

Three commercial enzyme preparations were used: α -amylase (EC 3.2.1.1; ≥ 100 U/mL, Sigma-Aldrich), endo-cellulase (EC 3.2.1.4; ≥ 100 U/mL, Sigma-Aldrich), and alkaline protease (EC 3.4.21.62; ≥ 2.4 AU/g, Novozymes). Enzyme activity was confirmed prior to use by standard spectrophotometric assays (DNS method for amylase/cellulase; azocasein method for protease; [14]).

Each enzyme was dissolved in sterile 50 mM phosphate buffer (pH 7.0) to prepare 100 U/mL stock solutions. Working solutions (10 U/mL final concentration) were prepared fresh daily by volumetric dilution into the respective matrix and stored on ice until use.

Justification of enzyme concentration: A final enzyme concentration of 10 U/mL was selected as a standardised proof-of-concept exposure level for evaluating the potential contribution of hydrolytic enzymes to coliform suppression in detergent-impacted matrices. The selected concentration falls within the activity range commonly employed in laboratory-scale investigations of enzymatic antimicrobial, antibiofilm, and biocatalytic processes [14–19]. No formal dose-response optimisation was conducted prior to experimentation; therefore, the selected concentration should not be interpreted as an optimal operational dose. Rather, it was intended to provide a consistent basis for comparing the relative performance of amylase, cellulase, protease, and detergent-associated treatments under identical experimental conditions.

Twelve treatment groups were established (six wastewater, six soil extract), each prepared in triplicate (n = 3), yielding 36 incubation vessels. Experimental groups are summarised in Table 2.

Each 500 mL vessel received 450 mL of matrix. For single-enzyme treatments, 50 mL of a 100 U/mL stock solution was added (final concentration: 10 U/mL). For detergent treatments (D+E and D–E), 1 g/L commercial liquid detergent (Active Co.) was added to the enzyme-containing formulation (D+E), which included amylase, cellulase, and protease; the enzyme-free formulation (D–E)

contained only surfactants and adjuvants. This design isolates enzymatic effects from surfactant-mediated antimicrobial activity, enabling direct comparison of individual enzyme contributions. Untreated controls received no additives. The overall experimental design and workflow are illustrated in Fig. 1.

Table 2. Experimental treatment groups.

No.	Treatment group	Matrix	Treatment	Enzyme conc. (U/mL)	Replicates (n)
1	WW-AMY	Wastewater	Amylase	10	3
2	WW-CEL	Wastewater	Cellulase	10	3
3	WW-PRO	Wastewater	Protease	10	3
4	WW-D+E	Wastewater	Commercial detergent + enzymes	10	3
5	WW-D-E	Wastewater	Detergent, surfactant only	—	3
6	WW-CTL	Wastewater	Untreated control	—	3
7	SOIL-AMY	Soil extract	Amylase	10	3
8	SOIL-CEL	Soil extract	Cellulase	10	3
9	SOIL-PRO	Soil extract	Protease	10	3
10	SOIL-D+E	Soil extract	Commercial detergent + enzymes	10	3
11	SOIL-D-E	Soil extract	Detergent, surfactant only	—	3
12	SOIL-CTL	Soil extract	Untreated control	—	3

• All groups were incubated aerobically at 25 ± 1°C for 7 days with orbital shaking at 150 rpm.

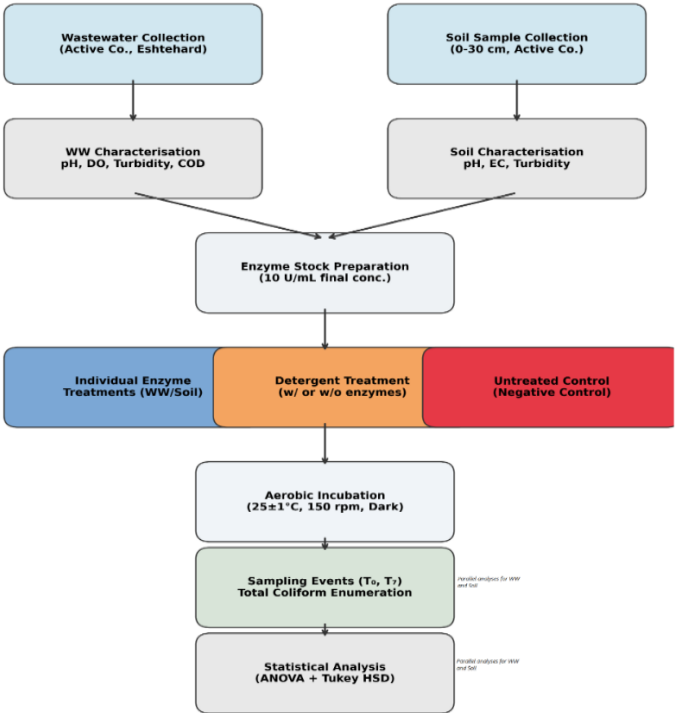


Fig. 1. Experimental design and workflow.

Schematic representation of the study design, including sample collection, enzyme stock preparation, treatment grouping, incubation conditions, and coliform enumeration. Parallel analyses were conducted on industrial wastewater and soil aqueous extracts to ensure a distinct evaluation of enzyme-mediated coliform reduction.

2.4. Incubation conditions

All vessels were incubated aerobically at 25 ± 1 °C under continuous orbital shaking (150 rpm) in a temperature-controlled incubator shaker (Innova 44 R, New Brunswick Scientific) in the dark. Sampling occurred at two time points: immediately upon mixing (T₀) and after 7 days (T₇). At each event, 10 mL was withdrawn aseptically from each vessel for coliform analysis. Physicochemical parameters (pH, DO, temperature) were recorded at both time points.

2.5. Total coliform enumeration by most probable number method

2.5.1. Serial dilution

At T₀ and T₇, 10 mL aliquots were serially diluted in sterile phosphate-buffered saline (PBS; pH 7.0) to yield dilutions from 10⁻¹ to 10⁻⁵. For samples exhibiting high initial coliform abundance (≥ 24 MPN/100 mL), serial dilutions were extended to 10⁻⁶ to ensure quantifiable tube combinations within the standard MPN range.

2.5.2. MPN assay

Total coliforms were quantified using the five-tube, three-dilution MPN method (APHA 9221-B). Five replicate tubes (10 mL Lauryl Tryptose broth each, with inverted Durham tubes) were inoculated with 1 mL of diluted sample and incubated at $35 \pm 0.5^\circ\text{C}$ for 48 ± 3 h. Gas production in the Durham tube or turbidity was recorded as presumptive positive. Presumptive positives were confirmed by subculture into Brilliant Green Lactose Bile broth ($35 \pm 0.5^\circ\text{C}$, 48 ± 3 h); only gas-producing tubes were counted.

MPN determination: MPN indices were determined using standard MPN tables with 95% confidence intervals from tabulated values. This approach avoids uncertainties inherent in mathematical approximations. Results below the detection limit (< 0.03 MPN/100 mL) reflect zero positive tubes across all dilutions. All analyses were conducted in the ISO/IEC 17025-accredited Environmental Engineering Laboratory, University of Tehran.

2.5.3. Quality assurance and quality control

QA/QC procedures included: (I) analytical blanks (PBS only) processed with each sample batch; (II) positive control strain (*E. coli* ATCC 25922, $\geq 10^6$ CFU/mL) in each MPN run; (III) sterilisation of all glassware and media by autoclaving; (IV) coliform enumeration within 6 h of sample withdrawal (APHA holding-time compliance); and (V) blinded analysis of all tube scoring by independent laboratory personnel to eliminate observer bias.

2.6. Calculation of coliform reduction efficiency

Coliform reduction efficiency was calculated as Eq. 1.

$$R (\%) = \left[\frac{C_0 - C_7}{C_0} \right] \times 100 \quad (1)$$

where C_0 and C_7 represent total coliform counts (MPN/100 mL) measured at T_0 and T_7 , respectively. To account for unequal baseline coliform concentrations among treatment groups, a log-reduction endpoint was also calculated for each replicate as follows: $\text{Log reduction} = \log_{10}^{C_0} - \log_{10}^{C_7}$, where C_0 and C_7 represent detection-limit-adjusted MPN values at T_0 and T_7 , respectively. Values below the detection limit were substituted with 0.03 MPN/100 mL prior to log transformation. This log-reduction metric was calculated as a supplementary measure to account for baseline variability among treatment groups, whereas primary statistical comparisons were performed using $\log_{10}$ -transformed T_7 coliform counts. Percentage reduction was retained solely as a descriptive measure of apparent removal efficiency.

2.7. Statistical analysis

All MPN values were $\log_{10}$ -transformed before analysis. Normality was assessed using the Shapiro-Wilk test ($\alpha = 0.05$), and homogeneity of variances was evaluated using Levene's test ($\alpha = 0.05$).

Because baseline (T_0) coliform concentrations differed among treatment groups, percentage reduction values were used only as descriptive indicators and were not used as the primary basis for statistical inference. Primary between-treatment comparisons were instead performed using $\log_{10}$ -transformed T_7 coliform counts, with absolute remaining counts (MPN/100 mL) reported as supporting endpoint data. This approach directly evaluates residual coliform abundance after treatment while avoiding the instability of percentage reduction estimates caused by unequal baseline values and proximity to the analytical detection limit.

Differences in $\log_{10}^{\text{transformed endpoint values}}$ counts among treatment groups were assessed using one-way analysis of variance (ANOVA), followed by Tukey's honestly significant difference (HSD) post hoc test ($\alpha = 0.05$). Where ANOVA assumptions were violated, the non-parametric Kruskal-Wallis H-test was applied, followed by Bonferroni-adjusted Mann-Whitney U pairwise comparisons.

Effect sizes were reported as partial eta-squared (η^2). Results from wastewater and soil extract groups were analysed separately because of their fundamentally different physicochemical characteristics, microbial ecology, and sorptive behaviour [22]. All analyses were performed using SPSS Statistics v26.0 (IBM Corp.), and figures were generated using GraphPad Prism v9.0. Data are presented as mean $\pm$ SD ($n = 3$), and statistical significance was defined as $p < 0.05$ (two-tailed).

The observed variation in treatment-group T_0 coliform concentrations, most notably the higher initial count recorded for WW-CEL (24 MPN/100 mL) relative to WW-CTL (4.6 MPN/100 mL), is consistent with enzymatic disruption of cellular aggregates or flocs immediately upon cellulase addition, potentially releasing previously non-culturable cells into suspension and thereby increasing the apparent enumerable coliform density at T_0 [9, 18]. Because primary statistical inference in this study is based on $\log_{10}$ -transformed T_7 endpoint counts rather than on percentage reduction from baseline, this inter-group T_0 variability does not compromise the validity of between-treatment comparisons; residual coliform abundance at T_7 remains the principal outcome measure, independent of initial concentration differences.

3. Results

3.1. Physicochemical baseline characteristics

Physicochemical parameters at T_0 are summarised in Table 1. Industrial wastewater exhibited near-neutral pH (7.2 ± 0.3), low dissolved oxygen (2.4 ± 0.6 mg/L), and high organic loading (COD $1,240 \pm 180$ mg/L), consistent with enzyme-containing detergent effluent. Anionic surfactant concentration (48 ± 9 mg/L LAS) exceeded typical inhibitory thresholds for biological treatment [1]. Soil aqueous extract showed lower organic loading and turbidity (COD N/D; turbidity 38 ± 11 NTU) and a near-neutral pH (6.8 ± 0.4), reflecting limited accumulation of process chemicals in the surrounding soil.

Overall, these baseline characteristics indicate two distinctly different exposure matrices: a high-strength, surfactant-rich wastewater environment with potential biological stressors, and a comparatively diluted soil aqueous extract with reduced organic and chemical loading.

3.2. Total coliform counts and reduction efficiency

Total coliform counts at T_0 and T_7 , $\log_{10}^{\text{transformed endpoint values}} [\log_{10}^{T_7}]$, reduction efficiencies (R%), and statistical significances are presented in Table 3. In accordance with the revised statistical framework, $\log_{10}^{T_7}$ values were used as the primary endpoint for between-treatment comparisons, while percentage reduction (R%) was retained solely as a descriptive indicator and not used for statistical inference. Visual comparisons of coliform abundance across treatment groups are presented in Fig. 2, while overall suppression patterns are summarised in Fig. 3. Wastewater and soil extract treatments were evaluated separately because of their fundamentally different physicochemical and microbiological characteristics [22].

Table 3. Total coliform abundance at baseline (T_0) and endpoint (T_7), $\log_{10}^{\text{transformed endpoint values}} [\log_{10}^{T_7}]$, reduction efficiency, and statistical significance across wastewater and soil extract treatment groups.

No.	Treatment group	Matrix	T_0 (MPN/100 mL)	T_7 (MPN/100 mL)	$\log_{10}^{T_7}$	Reduction (%)	P-value	Significance
1	WW-AMY	Wastewater	0.09	< 0.03	≤ -1.52	66.7	0.032	*
2	WW-CEL	Wastewater	24.0	< 0.03	≤ -1.52	99.9	< 0.001	***
3	WW-PRO	Wastewater	0.12	< 0.03	≤ -1.52	75.0	0.018	*
4	WW-D+E	Wastewater	24.0	0.19	-0.72	99.2	< 0.001	***
5	WW-D-E	Wastewater	0.24	0.03	-1.52	87.5	0.041	*
6	WW-CTL	Wastewater	4.60	0.53	-0.28	88.5	—	Reference
7	SOIL-CEL	Soil extract	0.06	< 0.03	≤ -1.52	51.6	0.043	*
8	SOIL-D+E	Soil extract	< 0.03	< 0.03	≤ -1.52	N/D	> 0.05	ns
9	SOIL-D-E	Soil extract	< 0.03	< 0.03	≤ -1.52	N/D	> 0.05	ns
10	SOIL-PRO	Soil extract	< 0.03	< 0.03	≤ -1.52	N/D	> 0.05	ns
11	SOIL-AMY	Soil extract	< 0.03	< 0.03	≤ -1.52	N/D	> 0.05	ns
12	SOIL-CTL	Soil extract	2.40	0.09	-1.05	96.2	—	Reference

* Values represent the mean of three independent replicates ($n = 3$). T_0 = immediately after treatment allocation and mixing (Day 0); T_7 = after 7 days of incubation. LOD = analytical detection limit (0.03 MPN/100 mL); N/D = not determinable (both T_0 and T_7 below the detection limit); ns = not significant ($p > 0.05$). $\log_{10}^{T_7}$ values were used as the primary statistical endpoint for between-treatment comparisons, while percentage reduction values are presented solely for descriptive interpretation of apparent removal efficiency. WW-CTL and SOIL-CTL served as untreated reference controls for wastewater and soil extract matrices, respectively.

3.2.1. Wastewater treatment groups

Substantial coliform decline was observed in the untreated wastewater control (WW-CTL), which decreased from 4.60 to 0.53 MPN/100 mL over the 7-day incubation period (88.5% reduction; Table 3). This pronounced background decline highlights the importance of accounting for environmental attenuation when interpreting treatment-associated responses. The observed trend may be consistent with conditions typically associated with industrial wastewater matrices, including nutrient depletion, low dissolved oxygen availability (2.4 ± 0.6 mg/L), and prolonged exposure to surfactant-containing environments [1, 4, 6, 22].

Given this control behaviour, treatment performance is described primarily in terms of residual endpoint abundance [$\log_{10}^{T_7}$] rather than percentage reduction values. WW-CEL and WW-D+E consistently exhibited the lowest residual coliform abundances at T_7 , with endpoint values reaching or approaching the analytical detection limit (Table 3). In comparison, WW-CTL remained at 0.53 MPN/100 mL, whereas WW-D-E reached 0.03 MPN/100 mL.

Although WW-CEL and WW-D+E showed descriptive reduction efficiencies of 99.9% and 99.2%, respectively, these values reflect both treatment-associated effects and the background decline observed in the untreated control. Accordingly, percentage reduction values were not used as the basis for treatment ranking.

An elevated T_0 value was observed in WW-CEL (24 MPN/100 mL) compared with WW-CTL (4.60 MPN/100 mL). Because treatment allocation preceded T_0 measurement, this difference may reflect variability associated with sample distribution following treatment addition. One possible explanation is the temporary dispersion of microbial aggregates following enzyme addition, which may increase the number of culturable units detected by the MPN assay. Similar enzyme-associated biofilm disruption and aggregate

dispersion phenomena have been reported in previous studies of hydrolytic enzyme systems [15, 17-19]. However, this interpretation remains hypothetical, as aggregate structure was not directly assessed in the present study.

The enzyme-free detergent treatment (WW-D-E) resulted in an 87.5% reduction, which was similar in magnitude to the decline observed in the untreated control (88.5%). Under the conditions examined, exogenous surfactant addition did not result in a clearly distinguishable change in coliform abundance relative to background attenuation based on the measured endpoint counts. However, this observation is limited to the specific concentration and exposure conditions applied in this study and should not be interpreted as evidence of intrinsic inefficacy of LAS. Previous studies have shown that surfactant behaviour is strongly dependent on concentration, microbial adaptation, matrix composition, and biodegradation dynamics [1, 4-6].

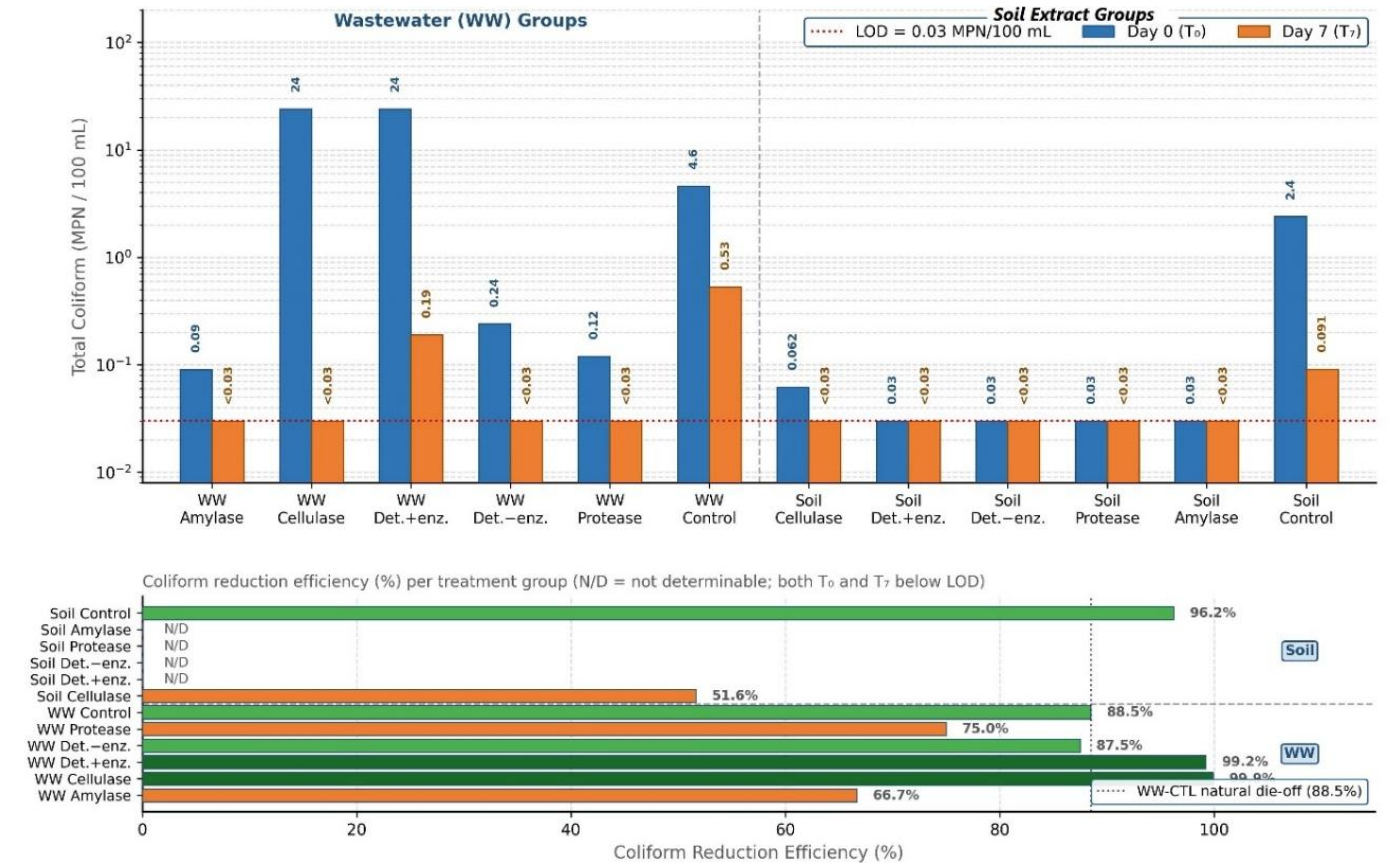


Fig. 2. Total coliform counts at T₀ and T₇ (Log Scale).

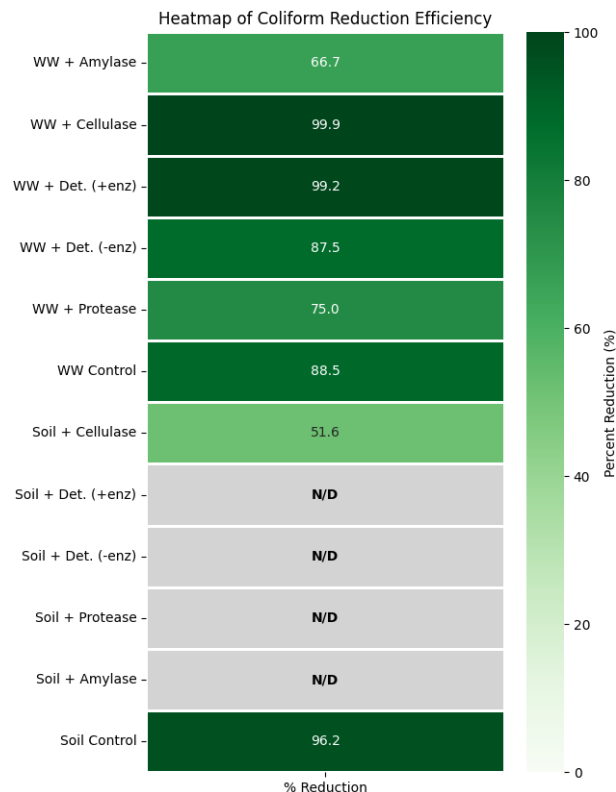


Fig. 3. Heatmap of coliform reduction efficiency.

3.2.2. Soil extract treatment groups

In the soil extract matrix, the majority of treatment groups exhibited baseline coliform counts at or below the LOD (0.030 MPN/100 mL; Fig. 2), precluding calculation of meaningful reduction efficiencies (designated N/D for SOIL-D+E, SOIL-D-E, SOIL-PRO, and SOIL-AMY; Table 3).

The fixed 10 U/mL enzyme dosage was standardized to address high-microbial-load wastewater scenarios (worst-case dosing strategy), resulting in a relative excess of enzymatic activity in the lower-load soil matrix, leading to a “floor effect” in which measured coliform abundance approached the analytical detection limit rather than reflecting treatment-driven dynamics. Only SOIL-CEL yielded a quantifiable baseline count ($T_0 = 0.06$ MPN/100 mL), which declined to <0.03 MPN/100 mL at T_7 (51.6% reduction; $p = 0.043$; Table 3 and Fig. 2). The soil control (SOIL-CTL) exhibited 96.2% natural reduction ($T_0 = 2.40$; $T_7 = 0.09$ MPN/100 mL), indicating substantial background attenuation under the applied incubation conditions.

Logarithmic bar plot displaying MPN/100 mL ($\log_{10}$ scale, y-axis) for all 12 treatment groups at both sampling time points (T_0 Day 0, T_7 Day 7, x-axis). Error bars represent $\pm$ SD ($n = 3$). Horizontal red dotted line indicates method detection limit (LOD = 0.030 MPN/100 mL). Percentages above each bar pair denote reduction efficiency (R%) relative to T_0 . Wastewater (WW) treatment groups shown in blue; soil extract (SOIL) groups in green. An asterisk indicates groups with T_0 below LOD values are presented for descriptive comparison; statistical inference is based on $\log_{10}^{T_7}$ endpoint analysis as described in Section 2.7.

Colour-gradient heatmap displaying magnitude of coliform suppression (0-100% reduction scale) across all 12 treatment groups from T_0 to T_7 . Treatment groups (WW and SOIL) listed on the y-axis; reduction percentage (0-100%) on the x-axis. Darker green shading indicates higher reduction efficiency; grey shading indicates N/D (not determinable: both T_0 and T_7 at or below LOD). Numeric labels within each cell display the exact reduction percentage. A horizontal dividing line visually separates wastewater and soil groups. Reduction values are shown for descriptive interpretation only and were not used for statistical comparisons. The temporal patterns of coliform decline in the wastewater treatment groups are illustrated in Fig. 4.

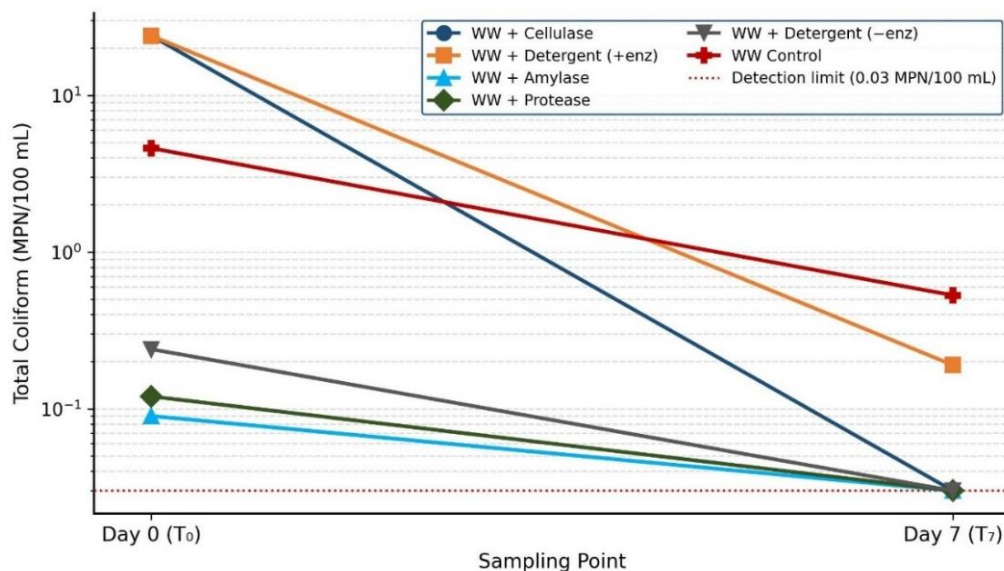


Fig. 4. Temporal decline in coliform counts wastewater treatment groups (log scale).

Semi-logarithmic plot displaying temporal dynamics of total coliform counts ($\log_{10}$ MPN/100 mL on y-axis) versus incubation time in days (0-7 days on x-axis) for six wastewater treatment groups: WW-AMY (amylase), WW-CEL (cellulase), WW-PRO (protease), WW-D+E (enzyme-supplemented detergent), WW-D-E (enzyme-free detergent), and WW-CTL (untreated control). Each treatment is represented by a distinct colour-coded line with error bars ($\pm$ SD, $n = 3$) at T_0 and T_7 . The horizontal red dotted line indicates method detection limit (LOD = 0.030 MPN/100 mL). Slope differences illustrate the rate of coliform decline under each treatment regime. Temporal trends are presented as observational patterns; endpoint statistical comparisons are based on $\log_{10}^{T_7}$ values.

The absence of detectable treatment effects in SOIL-D-E relative to SOIL-CTL may additionally reflect matrix-specific constraints on surfactant bioavailability, including possible adsorption of LAS onto soil particles and the potential for surfactant concentrations to fall below the critical micelle concentration (CMC) under the applied conditions [1, 5]; however, adsorption processes were not directly measured, and this interpretation is offered only as a plausible hypothesis.

4. Discussion

The present study evaluated the response of total coliform populations to hydrolytic enzyme treatments in detergent-impacted industrial wastewater and soil aqueous extracts under controlled laboratory conditions. A central finding was the substantial background attenuation observed in the untreated wastewater control, which exhibited an 88.5% decline in coliform abundance during the 7-day incubation period. This observation indicates that a large proportion of the overall reduction occurred independently of enzyme addition and highlights the importance of distinguishing treatment-associated responses from natural environmental attenuation.

In accordance with the revised statistical framework, treatment performance was interpreted primarily using residual endpoint abundance $\log_{10} T_7$ and absolute remaining coliform counts rather than percentage reduction values alone. Under these conditions, WW-CEL and WW-D+E exhibited the lowest endpoint coliform abundances among the wastewater treatment groups, whereas the enzyme-free detergent treatment (WW-D-E) produced an endpoint response that was comparable to the untreated control.

Although descriptive reduction efficiencies exceeded 99% in WW-CEL and WW-D+E, these values incorporate both treatment-associated effects and the substantial background decline observed in WW-CTL. Consequently, the findings are more appropriately interpreted as evidence of an incremental treatment-associated response superimposed upon a strong natural attenuation process rather than as a direct measure of enzymatic contribution alone. Relative to the untreated control, the descriptive reduction efficiencies observed for WW-CEL and WW-D+E exceeded the control response by approximately 11.4% and 10.7%, respectively. These values are presented only as descriptive indicators and should not be interpreted as formal estimates of enzymatic effect size.

4.1. Natural attenuation and wastewater responses

A prominent finding of the present study was the substantial decline observed in the untreated wastewater control (WW-CTL), which exhibited an 88.5% reduction in coliform abundance over the 7-day incubation period. This result indicates that environmental attenuation constituted a major component of the overall response and therefore required explicit consideration when interpreting treatment-associated effects. The observed decline is consistent with multiple stressors known to influence microbial persistence in industrial wastewater environments, including nutrient depletion during prolonged incubation, low dissolved oxygen availability (2.4 ± 0.6 mg/L), and exposure to surfactant-containing matrices. Similar interactions between surfactant-rich environments and microbial community dynamics have been reported in recent wastewater studies [1, 4, 6, 22].

Because baseline coliform concentrations differed among treatment groups, treatment performance was evaluated primarily using residual endpoint abundance [$\log_{10} T_7$] and absolute remaining counts rather than percentage reduction values alone. Under this framework, WW-CEL and WW-D+E exhibited the lowest residual coliform abundances at the end of the incubation period, with endpoint values reaching or approaching the analytical detection limit. These observations indicate that enzyme-containing treatments were associated with lower endpoint coliform abundance than the untreated control under the experimental conditions evaluated.

The difference between gross reduction and treatment-associated response is particularly important in the present dataset. Although descriptive reductions exceeded 99% in WW-CEL and WW-D+E, a substantial proportion of this decline was also observed in the untreated control. Consequently, the findings are more appropriately interpreted as an incremental response superimposed upon a strong background attenuation process. Relative to WW-CTL, the additional reduction associated with WW-CEL corresponded to approximately 11.4% beyond the control baseline, indicating a measurable but considerably smaller effect than implied by percentage reduction values alone.

An additional observation was the elevated T_0 count measured in WW-CEL relative to WW-CTL. Because treatment allocation occurred before baseline enumeration, this difference may not necessarily represent a true difference in the initial microbial burden of the wastewater. One possible explanation is the partial dispersion of microbial aggregates immediately following enzyme addition, thereby increasing the number of culturable units detected by the MPN assay. Hydrolytic enzymes have previously been reported to disrupt extracellular polymeric substances, biofilm structures, and microbial aggregates, potentially increasing the recovery of individual culturable cells [15, 17–19]. However, aggregate structure was not directly evaluated in the present study, and this explanation should therefore be regarded as a plausible hypothesis rather than a confirmed mechanism.

The response observed in the enzyme-free detergent treatment (WW-D-E) further highlights the importance of matrix context when interpreting surfactant effects. Although WW-D-E exhibited substantial coliform decline, the magnitude of reduction was similar to that observed in the untreated control. Under the concentration, exposure duration, and wastewater conditions examined here, exogenous surfactant addition did not produce a clearly distinguishable endpoint response beyond the background attenuation process. This finding should not be interpreted as evidence that LAS is intrinsically ineffective as an antimicrobial stressor. Previous studies have demonstrated that surfactant behaviour is strongly influenced by concentration, microbial adaptation, biodegradation pathways, and wastewater matrix characteristics [1, 4–6]. Therefore, the absence of a detectable additional effect in the present study is most appropriately interpreted as being specific to the experimental conditions evaluated rather than as a general property of LAS-containing systems.

4.2. Matrix-dependent behaviour: wastewater and soil extract responses

The contrasting outcomes between wastewater and soil extract matrices reflect fundamental differences in microbial abundance, substrate availability, and matrix characteristics. Previous studies have shown that microbial responses in environmental systems are strongly shaped by matrix-specific physicochemical conditions, resource availability, and community structure, often resulting in markedly different treatment outcomes despite identical interventions [12, 21, 22]. In the wastewater matrix, relatively higher baseline coliform abundances were observed ($T_0 = 4.60$ MPN/100 mL for the untreated control; up to 24.0 MPN/100 mL in WW-CEL and WW-D+E), whereas most soil extract treatments exhibited coliform counts at or below the analytical detection limit at baseline. Consequently, interpretation of treatment-associated responses was considerably more constrained in the soil matrix than in wastewater, particularly because statistical comparisons relied on endpoint abundance rather than percentage reduction values.

The predominance of sub-detection-limit counts in soil extract treatments is consistent with a floor effect, whereby measured

coliform abundance approached the analytical detection limit irrespective of treatment. Under these conditions, the ability to distinguish treatment-associated responses from background variability becomes inherently limited. The fixed enzyme dosage (10 U/mL) was originally selected as a standardised proof-of-concept exposure level for detergent wastewater and may therefore have exceeded the effective requirement for the substantially lower microbial loads present in the soil extract matrix. Similar substrate-limited responses have been reported in environmental microbial systems, where treatment effects become increasingly difficult to resolve as target populations approach low-abundance thresholds [12, 22].

In addition to low baseline microbial abundance, matrix-specific interactions may also have influenced treatment performance. Sorption processes are known to alter the environmental availability of biologically active compounds in heterogeneous solid-containing systems. Studies examining sludge and particulate-rich environmental matrices have demonstrated that adsorption phenomena can substantially influence microbial accessibility to contaminants and bioactive agents [7, 8]. Consequently, enzymes and surfactants introduced into the soil extract may have undergone partial adsorption onto soil particles, thereby reducing their effective bioavailability. Although adsorption processes were not directly measured in the present study, such mechanisms provide a plausible explanation for the limited differentiation observed among soil treatments.

The sole quantifiable soil treatment, SOIL-CEL ($T_0 = 0.06$ MPN/100 mL; 51.6% reduction, $p = 0.043$), exhibited a decline to below the analytical detection limit at T_7 . However, given the extremely low baseline abundance and the proximity of both measurements to the detection limit, this observation should be interpreted cautiously and not as definitive evidence of superior cellulase performance in the soil matrix.

The untreated soil control (SOIL-CTL) exhibited a 96.2% reduction over the incubation period ($T_0 = 2.40$ MPN/100 mL; $T_7 = 0.09$ MPN/100 mL), indicating substantial background attenuation under the experimental conditions. This observation suggests that intrinsic matrix properties exerted a dominant influence on endpoint coliform abundance. In addition, adsorption of surfactants onto soil particles may have reduced the freely available fraction of LAS, thereby limiting exposure of microorganisms to surfactant-associated stress. Such matrix-dependent reductions in bioavailability have been widely reported for contaminants and biologically active compounds in complex environmental systems [7, 8, 21]. Consequently, the present data do not support strong conclusions regarding enzyme-specific effects in soil extracts, but rather indicate that treatment-associated responses were difficult to resolve under low-load conditions approaching the analytical detection limit.

4.3. Practical implications for industrial wastewater treatment

The findings of the present study suggest that hydrolytic enzyme addition may influence endpoint coliform abundance in detergent-impacted wastewater under the experimental conditions evaluated. Because substantial background attenuation was observed in the untreated wastewater control (88.5% reduction), interpretation of treatment performance should focus on the incremental response observed beyond this baseline rather than on gross reduction values alone.

Within this context, WW-CEL and WW-D+E exhibited the lowest residual coliform abundances at T_7 , indicating that enzyme-containing treatments may contribute to additional suppression beyond the attenuation observed in untreated wastewater. However, the magnitude of this contribution should be interpreted in relation to the strong background decline observed in the control rather than as an independent measure of treatment efficacy.

These observations support the potential value of hydrolytic enzymes as supplementary components within wastewater treatment strategies rather than as stand-alone replacements for established treatment or disinfection processes. The present study was designed as a proof-of-concept investigation and did not evaluate regulatory compliance, treatment reliability under operational conditions, pathogen inactivation kinetics, or performance at pilot or full scale. Consequently, direct comparisons with conventional disinfection technologies such as chlorination or ultraviolet treatment are beyond the scope of the current dataset.

The comparatively low endpoint abundances observed in WW-CEL and WW-D+E nevertheless suggest that further investigation of enzyme-assisted treatment approaches may be warranted, particularly in detergent-rich industrial effluents where surfactants, organic loading, and microbial stressors coexist. Similar process-specific optimisation strategies have been widely advocated in industrial biocatalysis and environmental biotechnology applications [4, 14].

More broadly, the results emphasise the importance of distinguishing treatment-associated responses from natural attenuation processes when evaluating biological interventions in environmental systems. Adoption of endpoint-based statistical comparisons, together with explicit consideration of control performance, may provide a more conservative and transparent framework for assessing future enzyme-assisted wastewater treatment strategies.

4.4. Limitations and future research directions

The attainment of coliform counts at or below the analytical detection limit ($LOD = 0.03$ MPN/100 mL) in several treatment groups demonstrates the limited resolution of cultivation-based enumeration methods when endpoint abundances approach the detection threshold. While low endpoint counts were observed in WW-CEL and WW-D+E, measurements near the detection limit inherently constrain discrimination among highly suppressed treatment groups. This limitation was particularly evident in the soil extract matrix, where several treatments exhibited baseline and endpoint counts at or below the analytical detection limit.

An additional limitation relates to the use of the Most Probable Number (MPN) method, which quantifies culturable coliforms rather than directly measuring cellular damage, physiological stress, or mechanisms of inactivation. Consequently, the present study

cannot determine whether the observed reductions reflect cell death, loss of culturability, aggregate disruption, or a combination of these processes.

Future investigations could improve mechanistic resolution by integrating cultivation-based methods with complementary molecular and cell-viability approaches, including quantitative PCR, fluorescence-based viability assays, and microscopic assessment of aggregate structure. Such methods would help distinguish between reductions in culturability and broader changes in microbial physiological status.

The experimental design was based on two sampling points (T_0 and T_7), which allowed evaluation of endpoint responses but did not permit detailed assessment of treatment kinetics. Additional intermediate sampling intervals would provide insight into the temporal dynamics of coliform attenuation, potential transient effects, and the possibility of microbial recovery during prolonged incubation. Likewise, longer observation periods would help determine the persistence of treatment-associated responses beyond the initial 7-day incubation period.

A further limitation concerns the use of a single enzyme concentration (10 U/mL), which was intentionally selected as a standardised proof-of-concept exposure level rather than an optimised operational dose. Because no dose-response analysis was performed, the minimum effective concentration and the optimal treatment range remain unknown. Future studies should therefore evaluate multiple enzyme concentrations across matrices with differing microbial abundances and physicochemical characteristics. Such work would facilitate assessment of treatment efficiency, operational feasibility, and matrix-specific performance under environmentally relevant conditions.

Finally, the marked differences in baseline coliform abundance among some wastewater treatment groups highlight the challenges associated with environmental microbial datasets. Although the revised statistical framework relied primarily on $\log_{10}$ -transformed endpoint counts and absolute residual abundance rather than percentage reduction values, future studies would benefit from larger sample sizes, increased replication, and experimental designs specifically developed to minimise baseline heterogeneity among treatment groups.

5. Conclusions

This study investigated the response of total coliform populations to hydrolytic enzyme treatments in detergent-impacted wastewater and soil aqueous extracts under controlled laboratory conditions. A major finding was the substantial background attenuation observed in the untreated wastewater control, indicating that natural environmental decay represented a significant component of the overall reduction observed during incubation.

When treatment performance was evaluated using residual endpoint abundance [$\log_{10} T_7$] and absolute remaining coliform counts, enzyme-containing treatments were associated with lower endpoint coliform abundances than the untreated control in the wastewater matrix. Among the evaluated treatments, cellulase-containing systems exhibited the lowest residual coliform abundance under the experimental conditions investigated.

The findings further indicate that, under the tested conditions, addition of an enzyme-free LAS-containing detergent formulation did not produce a clearly distinguishable response beyond the background attenuation observed in the untreated control. This observation should be interpreted within the context of the specific matrix characteristics, concentration levels, and exposure conditions examined.

Overall, the results suggest that hydrolytic enzymes may contribute to incremental reductions in coliform abundance in detergent-impacted wastewater, while natural attenuation processes remain a dominant component of the overall response. These findings provide preliminary evidence supporting further evaluation of enzyme-assisted approaches for the management of microbiological contamination in surfactant-rich industrial wastewater systems.

Statements & declarations

Author contributions

Siamak Boudaghpour: Investigation, Supervision, Resources, Writing - Original Draft.

Amirali Bahadori: Formal analysis, Project administration, Writing - Review & Editing.

Ehsan Qamari: Conceptualization, Methodology, Resources, Writing - Original Draft.

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Data availability

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

Declarations

The authors have no conflicts of interest to declare that are relevant to the content of this article.

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Evaluating Effective AI Models for Surface and Groundwater Quality Prediction: A Review

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ABSTRACT

Artificial intelligence (AI) techniques have become important tools for predicting and assessing water quality because they can capture nonlinear relationships among complex environmental variables. This review systematically analyzes AI-based approaches for surface water and groundwater quality prediction and identifies factors that influence model performance. Following PRISMA guidelines, we conducted a systematic review using five scientific databases, screening 224 records and including 35 peer-reviewed studies. We analyzed the reviewed studies by AI model type, water resource category, input parameters, dataset characteristics, hydrological conditions, and prediction performance. The results indicate that hybrid and ensemble models generally outperform standalone algorithms by improving feature extraction, reducing uncertainty, and better capturing nonlinear relationships. GPR showed high accuracy for WQI prediction with uncertainty below 2%, RF-SVM achieved strong performance for BOD prediction ($R^2 = 0.908$), and LSTM-based and metaheuristic models improved DO prediction. Despite their high accuracy, AI models still face challenges related to interpretability, data availability, and regional transferability. This review compares AI model suitability under different hydrological conditions and proposes future directions in explainable AI, uncertainty quantification, and adaptive real-time water quality monitoring.

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1. Introduction

Water is a limited, essential natural resource that plays a fundamental role in sustaining life, ecosystems, and socio-economic development. As the world population continues to rise rapidly (estimated to exceed 9 billion by 2050), demand for clean, accessible water is rising at a record rate. This increased demand puts immense pressure on existing water supplies, some of which are already stressed by pollution, global warming, and overuse. Maintaining water quality is therefore crucial not only for public health but also for environmental protection and sustainable development. However, traditional methods of quantifying water quality, while effective, are slow, resource-intensive, and limited in providing real-time or predictive results.

In these situations, incorporating artificial intelligence (AI) into water quality measurement has emerged as a promising, advanced approach. AI technologies can handle large, complex sensor and remote-sensing datasets to provide more accurate, efficient, and timely evaluations of water conditions. AI can also identify hidden patterns, predict contamination events, and support decision-making in water resource management. These are potential applications, but a major scientific gap remains in

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systematically using and testing AI models across a broad spectrum of hydrological conditions. Research that addresses these differences is critical to unlocking AI's full potential for managing water quality and building scalable, adaptive, and robust systems. Filling this research gap not only improves our technological solutions for water problems but also supports global sustainability goals by protecting one of the world's greatest assets.

Elkiran et al. [1] presented single-step and multi-step prediction models with AI and ensemble techniques for dissolved oxygen (DO) at three Yamuna River stations. They used AI models such as back propagation neural network (BPNN), adaptive neuro-fuzzy inference system (ANFIS), and support vector machine (SVM), along with auto-regressive integrated moving average (ARIMA), and each yielded better results than the others at different stations. Additionally, ensemble techniques such as simple average ensemble (SAE), weighted average ensemble (WAE), and neural network ensemble (NNE) further improved prediction accuracy and showed that NNE was the most robust. The combination of various algorithms with ensemble methods in the suggested study offers a way to improve predictions for DO and other water quality parameters.

Hameed et al. [2] applied an artificial neural network (ANN) to predict water quality index (WQI) from tropical environments using monthly water quality measurements collected over ten years from 2001 to 2010. The researchers compared two models: BPNN and radial basis function neural network (RBFNN). ANN predicted the WQI satisfactorily, but RBFNN performed best. This study identifies RBFNN as a powerful tool for river water quality assessments, with strong potential to model real systems involving nonlinear dynamics and stochastic behavior for each water quality parameter.

Najah Ahmed et al. [3] evaluated the enhanced wavelet de-noising technique (WDT)-ANFIS for predicting water quality parameters influenced by urbanization. Considering the Johor River dataset, this work evaluates the accuracy and reliability of WDT-ANFIS, ANFIS, multi-layer perceptron (MLP)-ANN, and radial basis function neural networks (RBF-ANN). The former was found to be the best among these for simulating complex and dynamic processes. Of the varied scenarios studied, scenario 2, which included the data from the upstream station, showed the best prediction. The study further opens up possibilities for WDT-ANFIS to make river basin management and policy-making more effective through improved water-quality forecasting.

Ahmed et al. [4] used machine learning (ML) to forecast water quality with the fewest parameters and used 663 samples obtained from Rawal Lake in Pakistan. Their results showed that polynomial regression and gradient boosting (GB) were highly successful in predicting WQI, while the MLP algorithm performed best for water quality class (WQC). Their work, therefore, has great utility for assessing water quality in the subject models using the mentioned ML techniques.

Azad et al. [5] investigated genetic algorithm (GA), differential evolution (DE), and ant colony optimization (ACO) for continuous domains (ACOR) evolutionary algorithms for optimization of the ANFIS model to predict the water quality parameters of irrigation and drinking water, namely electrical conductivity (EC), sodium absorption ratio (SAR), and total hardness (TH). The results showed that the integrated models were highly accurate, and ANFIS-DE performed more efficiently than the other methods. In the current research, these algorithms significantly reduced uncertainty and overfitting in the ANFIS model during training and validation. Among them, ANFIS-DE performed better in terms of performance measures and prediction accuracy. In a nutshell, the study showed that metaheuristic algorithms can effectively enhance ANFIS for predicting river water quality parameters.

Wang et al. [6] estimated the WQI by remote sensing data and algorithms in the Ebinur Lake watershed of China. Accordingly, WQI values ranged from 56.6133 to 2,886.5198, representing generally low water quality in the arid region. The coefficient of correlation (R) was high and increased with increasing orders of derivatives up to order 1.6, improving the predictive capability of the models. Hence, among all 22 generated models, the ratio index (RI), difference index (DI), and normalized difference index (NDI)-based models performed best at the 1.6-order derivative. Thus, this new algorithm shows strong potential for application in other semi-arid regions.

Granata et al. [7] applied an indirect method to estimate key effluent quality parameters. In this method, they considered several characteristics of an urban catchment: drainage area; percentages of residential, institutional, commercial, and industrial areas; open space areas; freeway and impervious area percentages; precipitation depth; and runoff. It compared two ML algorithms: support vector regression (SVR) and regression tree. Both models were robust and reliable and showed strong generalization capability. However, SVR outperformed the regression tree in the estimation of total suspended solids (TSS), total dissolved solids (TDS), and chemical oxygen demand (COD) based on the root mean square error (RMSE) and coefficient of determination (R^2). For biochemical oxygen demand (BOD), both models performed similarly. ML algorithms can also help estimate values for sizing treatment units using basin characteristics when direct measurements are not available. Their study showed this could be an effective methodology for real-time management of wastewater treatment plants and environmental planning.

Hridoy et al. [8] tested different machine learning models, such as extreme gradient boosting (XGB), random forest (RF), and SVM, for water quality classification and WQI prediction. The authors used grid search to tune model parameters and Shapley additive explanations (SHAP) to interpret predictions and identify key water-quality predictors. The tests found that ensemble machine learning models, especially the XGB model, provided outstanding classification results, with an accuracy of 99.65%. For WQI prediction, XGB demonstrated outstanding performance, with an R^2 of 0.9685. According to SHAP analysis, the main water quality predictors were DO and BOD, followed by turbidity, nitrate, and EC.

Li et al. [9] used machine learning models to predict ultrapure water quality one hour in advance using total organic carbon (TOC) and resistivity. They compared six approaches and chose Optuna-XGB for TOC prediction. The model performed well, with R^2 values of 0.9664 and 0.9670, respectively. We used SHAP and partial dependence plots to interpret the results and identify the major factors influencing water quality. Return water resistivity was the major factor in both cases, and we observed significant

nonlinear interactions between reverse osmosis rejection rate and operational conditions.

Mena et al. [10] presented an integrated framework for water quality assessment and regulatory compliance in river basins under hydrological uncertainty. The methodology combines a long short-term memory (LSTM) deep learning (DL) algorithm for streamflow forecasting, Monte Carlo simulation for uncertainty propagation, hydrological disaggregation through the HBV model, and process-based water quality modeling. The methodology was tested in the Júcar River basin in eastern Spain, characterized by high regulation, hydrological uncertainty, and water quality problems. The process-based water quality model simulates BOD, phosphorus, and ammonium concentrations. The framework reproduced the spatial and temporal patterns of water pollution and quantified uncertainty in hydrological forecasting. Accuracies in the chemical status classification reached 90% for BOD, 84.4% for phosphorus, and 85.3% for ammonium.

Due to the extensive utilization of AI techniques in predicting water parameters [11-14], and also because studies have shown that even one AI technique can be utilized in various aspects of water engineering [15], analyzing and classifying studies conducted in the field of water quality prediction via AI can help to achieve a better and more detailed horizontal understanding in this field.

2. Research method

In this article, to develop a suitable package of "AI application in water quality determination," we first defined a method for searching the latest studies in this field. Then, using keywords including "AI", "water quality", "water resources," etc., our survey began. We used the ScienceDirect, MDPI, Wiley Online Library, Nature, and Springer reference databases to access relevant research articles. Table 1 illustrates the data extraction and methodological characteristics. After receiving several articles on "artificial intelligence application in water quality determination", all non-English literature and conference abstracts were omitted. By reviewing each article with focus and precision on the points and keywords, we extracted a suitable abstract from each article. Finally, we matched and ordered the abstracts to prepare this article. All research articles are properly referenced. Fig. 1 demonstrates the PRISMA flow diagram of this review.

Table 1. Data extraction and methodological characteristics of the studies included in the systematic review.

Databases	ScienceDirect, MDPI, Wiley, Nature, Springer
Search period	e.g., exact years
Search terms	exact Boolean string
Initial records	224
Duplicates	57
Screened	148
Full-text assessed	56
Final studies	35
Inclusion criteria	explicit
Exclusion criteria	explicit

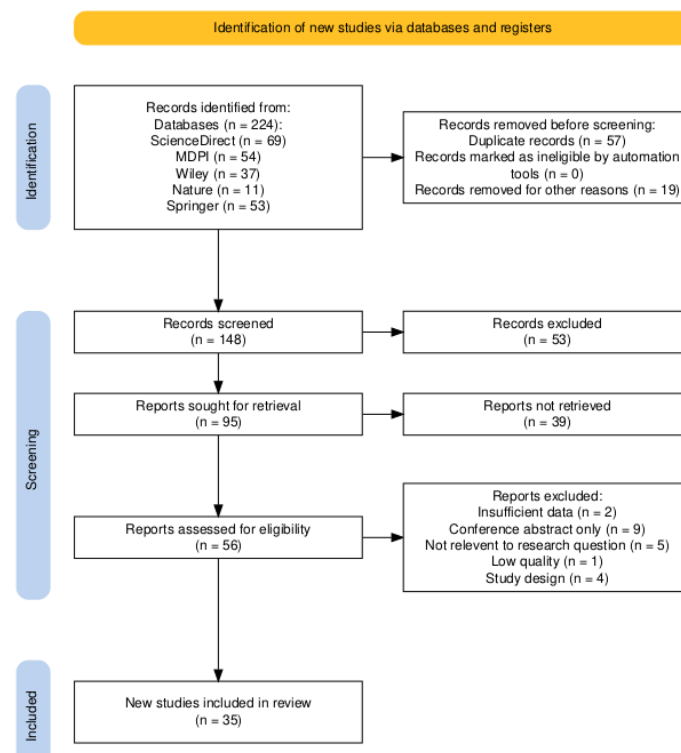


Fig. 1. PRISMA flow diagram.

This systematic review followed the preferred reporting items for systematic reviews and meta-analyses (PRISMA) guidelines. We conducted a comprehensive literature search across five scientific databases: ScienceDirect, MDPI, Wiley Online Library, Nature, and Springer. We defined the search period as 2017-2024 to focus on recent developments in artificial intelligence applications for water quality prediction. The search strategy was based on combinations of keywords and Boolean operators, including ("artificial intelligence" or "machine learning" or "deep learning" or "neural network") and ("water quality" or "water quality index" or "surface water" or "groundwater") and ("prediction" or "assessment" or "modeling"). The initial search identified 224 records. After removing 57 duplicates, we screened 148 articles for title and abstract relevance. We then assessed 56 articles through full-text review, and finally included 35 studies in this review. Studies were included if they (i) were published as peer-reviewed journal articles, (ii) were written in English, (iii) applied AI-based methods for water quality assessment or prediction, and (iv) provided sufficient information regarding model development and evaluation. We excluded conference papers, review articles, non-English publications, and studies unrelated to AI-based water quality prediction. The screening and selection process followed the PRISMA framework presented in Fig. 1.

Journal reputation and name did not influence article selection. We used article quality and topic alone as evaluation criteria to keep the review on course. Therefore, the selected articles share a theme and topic, enabling a more reliable, valid, and meaningful conclusion. We limited the articles to the most recent publication years. This ensured that recent studies support the results and minimized the possibility that the selected articles had already been reviewed in previous review articles. In addition, we selected articles with minimal geographic overlap to examine a wider range of regions.

3. Artificial intelligence application in water quality determination

3.1. Surface Water

Wang et al. [16] used water quality monitoring data from Shandong province, China. They devised an AI system to identify spatial and temporal variations in water quality and understand the relationships between water quality measures and sources of industrial pollution. In this respect, the method proposes an intelligent approach to assess complex relationships between pollution sources and water quality data and to investigate interactions among water quality indicators. It then employs cross-correlation analysis to map water quality indicators strongly correlated with industrial pollution sources. The findings align with real-world conditions observed in water quality and pollution sources. These maps can help identify any sudden changes in discharge due to pollution and locate industries that affect water quality. Second, the Apriori method identified high-frequency indicator pairs across monitoring stations, showing that water quality parameters vary spatially and change with industrial activity over time. Using LSTM, the neural network predicted how industrial pollution would affect future water quality. This will increase our ability to manage pollutant discharge and help enhance river basin water quality.

Nasir et al. [17] focused on water quality classification using ML techniques and proposed an intelligent, real-time approach to water quality monitoring. They extended this with different standard variants and ensemble models to conduct a comprehensive analysis. This dataset consists of over 1600 samples collected from various places in India, ranging from 2005 to 2014. The metadata parameters include DO, pH, conductivity, BOD, nitrate, fecal coliform, and total coliform. The ML classifiers included SVM, random forest (RF), logistic regression, categorical boosting (CATBoost), XGB, DT, and MLP. Performance metrics calculated from the confusion matrix included precision, sensitivity, F1 score, and accuracy. Precision-recall curves, receiver operating characteristic (ROC) curves, and average precision curves were also obtained. Among these, CATBoost achieved the highest maximum precision (94.51%), followed by the RF classifier (94%), then the MLP (88.6%), XGB (88.1%), DT (81.6%), SVM (80.7%), and logistic regression (72.9%). However, in the stacking ensemble model, various meta-classifiers, including DT and CATBoost, achieved 100% accuracy. Thus, CATBoost became the best classifier and the best stacking model, improving accuracy. Because of its strong performance metrics, the CATBoost classifier is a good tool to apply to different datasets. Such analyses can be further enriched in cooperation with experts in ML implementation and other fields, such as hardware implementation, internet of things (IoT), and biomedical research.

Bui et al. [18] examined the performance of 16 data-mining algorithms, including four standalone techniques such as RF, M5 Prime (M5P), random tree (RT), and reduced error pruning tree (REPT), and 12 hybrid versions, which are standalone combined with bagging, CV parameter selection (CVPS), and randomizable filtered classification (RFC) for prediction of WQI every month in a humid area in Mazandaran and Semnan provinces in northern Iran. In addition to developing new WQI prediction algorithms, the aim was to capture other aspects of water science in areas without regular water quality monitoring stations. Among the parameters, fecal coliform concentration was identified as the most influential factor in determining the WQI, followed by BOD, DO, EC, COD, turbidity, total solids (TS), and pH. Different combinations of the selected variables may affect model performance differently, and irregularities in model-input variation may occur across catchments. A high R value indicates good performance; otherwise, low R values reduce predictive power. Generally, hybrid algorithms somewhat outperformed the standalone algorithms, although the results differed in practice. For instance, the BA-RT model performed best, followed by RF, bagging-RF, etc. However, it performed poorly for extreme WQI values. Most algorithms tended to overestimate WQI, except RT, BA-RT, and CVPS-REPT. These will be especially valuable in catchments with limited monitoring networks, which are common in most developing countries because of the high cost of measurement. However, suitability varies by algorithm and cannot be generalized across all contexts. BA-RT is a highly promising tool for managing groundwater quality in the humid regions of northern Iran and may be less effective elsewhere.

Sajib et al. [19] evaluated the water quality for the Tongi canal in Bangladesh. They used an unweighted Root Mean Square

(RMS)-WQI model along with the eight ML algorithms, namely ensembles of trees (EN), Gaussian process regression (GPR), linear regression (LR), neural networks (NN), stepwise linear regression (SLR), SVM, regression of trees (TR), and XGB. We developed these models to predict the WQI score. We used various model evaluation techniques, including RMSE, Mean absolute error (MAE), R^2 , Nash-Sutcliffe efficiency (NSE), model efficiency factor (MEF), PERI, Z statistics, and Taylor statistics, to identify the most accurate model. We also performed sensitivity and uncertainty analyses using R^2 and Monte Carlo simulation (MCS) methods. The main findings are: (1) all parameters of water quality exceeded the guideline values prescribed by Bangladesh. (2) The WQI score showed the "poor" quality of water in the Tongi canal, and thus, it is not suitable for any purpose. GPR performed best across various metrics, including RMSE, MAE, R^2 , NSE, MEF, Z, and Taylor statistics, for predicting the WQI score. It performed well in evaluating the ecological state of urban waters. The uncertainty analysis showed that the RMS-WQI model reduces uncertainty to below 2% without key issues such as eclipsing or ambiguity. Therefore, the RMS-WQI model seems appropriate for assessing water quality in similar inland water bodies.

Yang [20] applied four nature-inspired stochastic search methods, including teaching-learning-based optimization, sine cosine algorithm (SCA), water cycle algorithm (WCA), and Electromagnetic Field Optimization (EFO), to train an ANN to predict DO in the Klamath River, Oregon, US. All methods could train the MLPNN to reliably capture DO behavior by properly tuning each algorithm's parameters. Therefore, the models could correctly predict DO under new environmental conditions. To determine the best hybrid model, we compared the models in terms of accuracy, complexity, and computation time. During training, the EFO algorithm required 30 times more iterations but completed the process much faster than the other three algorithms. It also produced the best results in the testing phase, as measured by RMSE, R, and NSE. Another merit of this model is that it uses fewer search agents to determine the best response. WCA-MLPNN was the second-best performer. In the final part of the study, we proposed two DO predictive models, with weights and biases tuned by the WCA and EFO algorithms. The superior models herein also outperformed several hybrids and conventional models reported in other studies, demonstrating improved practical DO prediction. This development may also help address the common problem of poor water quality in the study area.

Abba et al. [21] developed, at three stations along the course of the Yamuna River (Nizamuddin, Palla, and Udi-Chambal), applications for the estimation of WQI using four different single models: BPNN, ANFIS, SVR, and multiple linear regression (MLR), through four various model combinations by implementing these single models. We assessed predictive accuracy using statistical criteria such as R^2 , RMSE, and R. Nonlinear ensemble techniques further improved model performance. The quantitative analysis showed that, during validation at Nizamuddin station, ANFIS-M3 improved absolute error by 30%, 41%, and 53% compared with BPNN, SVR, and MLR, respectively. At the Palla station, the BPNN-M4 model reduced the absolute error by up to 4% compared with the MLR model, whereas the remaining models (SVR and ANFIS) showed only marginal improvements. Meanwhile, for the Udi (Chambal) station, the absolute error increased by about 2% and 3% for the BPNN-M3 model compared to the SVR and MLR models, respectively. Overall, the results indicated that the NNE method provides efficient WQI predictions at the three stations along the Yamuna River and can be considered a reliable prediction approach. However, although the ensemble method outperformed any single model, a single model can still provide efficient predictions. Overall, the study showed that, in ensemble techniques, combining the best outputs from different models yields better performance than any single model. Further research on other data intelligence models is recommended to improve WQI prediction. Finally, an ensemble technique developed in their work can be applied to various rivers across different regions. While it outperforms standalone models, there is still room for improvement.

Wu and Wang [22] proposed the ANN-wavelet transform (WT)-LSTM model to improve water quality prediction accuracy. This model integrates ANN, WT, and LSTM. The model uses water quality data from the Jinjiang River basin in China to predict water quality and analyze trends. Because some data are missing due to instrument failure, an ANN fills the gaps using time-series information. WT then decomposed and reconstructed the time series, filtering out the short-term random noise and improving the model's prediction accuracy for both out-of-sample data and future dynamic trends. This approach allowed for more effective short-term and long-term water quality predictions. Consequently, the ANN-WT-LSTM model outperformed ANN-LSTM and nonlinear autoregressive (NAR) neural network models across all metrics and significantly improved water quality prediction accuracy, which is important for water environment protection. They provide essential data support for water quality management decisions and have theoretical and practical significance for the sustainable development and environmental protection of riverine and reservoir areas. Although time series analysis has successfully predicted future reservoir water quality trends, most prior studies have limitations: they can predict only single-point water quality and do not account for spatial variation. To establish a comprehensive reservoir early warning system, multiple water-quality monitoring points are recommended. Integrating water quality prediction with GIS technology would capture temporal and spatial changes in water quality and lay the basis for a robust water quality early warning system.

Asadollah et al. [23] compare the performance of three ML models for WQI forecasting in the Lam Tsuen River, Hong Kong, using monthly water-quality measurements. These are standalone SVR and decision tree regression (DTR); the novel ensemble extra tree regression (ETR) algorithm; and various combinations of input parameters, M1 to M10. The results show that the ETR model, by taking ten input parameters, offers the best performance in WQI prediction. SVR performed similarly but slightly worse, and the DTR model performed the poorest, though it still predicted WQI fairly well. Remarkably, the ETR model predicted equally well with three inputs of BOD, turbidity, and phosphate. This suggests that WQI can be estimated accurately with limited monitoring, provided these three parameters are measured. The uncertainty analysis showed that WQI prediction is more sensitive to the model type (DTR, SVR, and ETR) rather than the input variables. With this in mind, ensemble learning models that combine weak learners such as DTs have higher accuracy than stand-alone decision-tree models or traditional AI models, including SVR.

More precisely, the ETR-M3 model, along with other models proposed in this work, can enable more efficient WQI computation for water quality engineers and decision-makers and, therefore, support real-time monitoring and better water management practices. However, a limitation is selecting only certain chemical and physical parameters, since the chosen sample may prove unsatisfactory.

Lu and Ma [24] developed two hybrid DT-based models, namely complete ensemble empirical mode decomposition with adaptive noise (CEEMDAN)-XGB and CEEMDAN-RF, for water quality prediction. In these models, CEEMDAN decomposes raw data with significant fluctuations further to enhance predictive performance for both XGB and RF. This study examines water quality in Gales Creek in the Tualatin River, Oregon, USA. We used data generated from May 1st to July 20th, 2019. The data were split into training and test sets in a 9:1 ratio. The models predict water temperature, DO, pH, specific conductance, turbidity, and fluorescent dissolved organic matter (FDOM), and their results are compared with benchmark models. In addition, the standard deviation of errors (SDE) is used to compare and check the model stability. Thus, the results show that the temperature, DO, and specific conductance tasks yield excellent performance; for these three, CEEMDAN-RF achieved the best performance, with respective mean absolute percentage errors (MAPE) of 0.69%, 1.05%, and 0.90%. However, pH value, turbidity, and FDOM performed best in CEEMDAN-XGB, with MAPEs of 0.27%, 14.94%, and 1.59%, respectively. (2) The overall performances of the RF, XGB, CEEMDAN-RF, CEEMDAN-XGB, particle swarm optimization (PSO)-SVM, RBFNN, least squares SVM, and LSTM models have average MAPEs of 6.41%, 4.60%, 3.90%, 3.71%, 8.96%, 6.93%, 6.96%, and 7.35%, respectively. Altogether, CEEMDAN-XGB has the best predictive performance among the compared models, followed by CEEMDAN-RF. (3) For each water quality indicator, the SDE of both CEEMDAN-RF and CEEMDAN-XGB is relatively lower than that of other benchmark models, indicating that these new models have better stability.

Al-Mukhtar and Al-Yaseen [25] studied and compared three AI models: ANFIS, ANN, and MLR to simulate and forecast TDS and EC in the Abu-Ziriq marsh in southern Iraq. The study evaluated all models using three performance indices: R, RMSE, and NSE. The ANFIS model fit the observed data better than the other methods. This superior performance was attributed to ANFIS's unique structure, which combines the advantages of fuzzy logic for function simplification and neural networks for self-learning, enabling it to handle data noise effectively. The authors recommend ANFIS as a reliable predictive tool for assessing water quality parameters, TDS and EC, in Iraqi marshes. They suggest that the approaches used in this work may be applied to other marshes, rivers, and basins to test their broader applicability. In addition, the tools and results developed herein provide a sound basis for managers, engineers, and decision-makers to improve the design, management, and decision-making related to water resources in Iraq and other similar regions.

Barzegar et al. [26] used four measurable water quality parameters, such as EC, pH, oxidation-reduction potential (ORP), and water temperature, for the development of DL models comprising LSTM and convolutional neural network (CNN), with the proposition of a new hybrid model called CNN-LSTM, including some traditional ML models such as SVR and DT to predict DO and chlorophyll-a (Chl-a) concentrations in Small Prespa Lake, Greece. We collected data every 15 minutes from June 1st, 2012, to May 31st, 2013. We transformed the samples into a supervised format and divided them into training and testing sets. We trained optimal models with their hyperparameters and evaluated performance on the test set using statistical metrics: R, RMSE, MAE, and their normalized forms, RMSR and MAE. Although the CNN and LSTM models performed similarly in predicting Chl-a, LSTM outperformed CNN in predicting DO concentration. Similarly, the hybrid CNN-LSTM model, which combined the strengths of both DL models, reduced the RMAE in DO prediction by 0.24% and 1.45% over the LSTM and CNN models, respectively. For Chl-a prediction, it reduced the RMAE by 6.41% and 6.48% over the LSTM and CNN models, respectively. The CNN model performed better at lower-level predictions, while LSTM models performed better at higher-level predictions of DO and Chl-a concentrations during the testing period. By integrating these models, the hybrid CNN-LSTM can more effectively capture both low- and high-level water quality variables for predicting DO concentrations.

Baek et al. [27] combined two DL models, CNN and LSTM, to simulate the water level and three water quality parameters of total nitrogen (TN), total phosphorus (TP), and TOC in the Nakdong River basin. The CNN model was used to simulate water level, and the LSTM model was used to simulate pollutant concentrations. The following are the findings: 1. The CNN modeled the simulation of water level well; in fact, the value of NSE was 0.933, which is an acceptable performance. Water levels were higher during the rainy season and lower during the dry season. The LSTM model for all the pollutants showed an NSE greater than 0.75 during both training and validation periods, which falls under the category "very good." The model accurately captured temporal variations in each pollutant. (3) TOC and TP exhibited a similar pattern in temporal variation, highly fluctuating in the rainy season, while TN increased in the spring season. The research concludes that the combined CNN-LSTM approach may be promising for accurate simulations of water levels and quality, considering better water sustainability and management strategies. This study examined three pollutants, but most process-based models can simulate a wider suite of water quality parameters, such as chlorophyll, algae, DO, and fecal bacteria.

Uddin et al. [28] attempted to assess water quality in the Bay of Bengal using XGB and an advanced WQI method. They improved their proposed generalized model by assessing performance and sensitivity in marine water evaluation. A wide variety of statistical parameters were used for the performance assessment and comparison of the model against the observed discharges, including prediction performance metrics: RMSE, mean squared error (MSE), and MAE; model sensitivity: R^2 ; and model efficiency: NSE and MEF. From the findings of their research, a summary is extracted as follows: most of the water quality indicators from the various sampling sites in the Bay of Bengal fell within guideline values, except for DO and molybdate reactive phosphorus (MRP), where DO was below the threshold value, while MRP was above the guideline limit. Both weighted WQM-WQI and unweighted RMS-WQI models classified water quality as generally "good" or "fair", except for site A, which, from the WQM-WQI model, was consistently ranked as good. Although the models produced different WQI scores, the water-quality rankings did not

differ significantly. Performance evaluation showed that the models incorporating the XGB algorithm were efficient in performing an accurate assessment of marine water quality; furthermore, all performance indicators-RMSE, MSE, and MAE-showed reliability in reducing model uncertainty below 1% with no significant issues. Sensitivity analysis showed strong agreement between model input and output ($R^2 = 1$), but the results were sensitive to the spatiotemporal resolution of water bodies. In addition, NSE and MEF indicated that both models efficiently calculated the correct WQI score. This investigation further highlights the efficiency of both weighted WQM-WQI and unweighted RMS-WQI models for evaluating water quality in the Bay of Bengal. These findings help enhance monitoring programs and the assessment of marine water using widely adopted approaches. However, the research was limited to only ten cross-sectional sampling sites. Expanding the analysis to include additional areas alongshore and using time-series data will help indicate the models' response over larger areas. Sensitivity testing across a range of water quality attributes over an extended period would also help generalize the models for the region.

Melesse et al. [29] examined the Babol-Rood River, an important source of irrigation and drinking water in its watershed; therefore, deterioration of the water supply may lead to significant environmental and social impacts. Monitoring and predicting the river's future status requires precise models, which are considered the backbone of resource conservation and management planning. This study is among the first contributions to evaluate the performance of two independent algorithms and eight new hybrid algorithms in salinity prediction in rivers. These algorithms included M5P, RF, bagging-M5P, bagging-RF, random subspace (RS)-M5P, RS-RF, random committee (RC)-M5P, RC-RF, additive regression (AR)-M5P, and AR-RF. We used monthly data over a continuous 36-year period to develop different input variable combinations and evaluate their performance. The results showed that TDS is the most critical variable for estimating EC, with a correlation of 0.99. We tested 10 input combinations, and TDS alone proved very important. This study showed that model structure and input variables strongly influence prediction accuracy. As shown by the validation criteria, M5P had higher predictive power than RF; therefore, hybrid algorithms incorporating M5P outperformed algorithms that used RF. M5P and its hybrids, except RC-M5P, predicted EC accurately even under extreme conditions. Hybrid algorithms generally improved standalone performance, except for the RC model, which reduced predictive power when hybridized with M5P and RF algorithms. Although all algorithms performed very well, AR-M5P was the best. From most to least accurate, the ranking was: AR-M5P, RS-M5P, bagging-M5P, M5P, RC-M5P, AR-RF, bagging-RF, RF, RS-RF, and RC-RF. These results apply only to this study and cannot be generalized to other areas or hydrological data, but AR-M5P will likely be among the most accurate algorithms. Overall, the hybrid models performed better than the individual models, and it is possible to calculate river water quality using advanced ML algorithms and multiple readily available input variables.

Leggesse et al. [30] investigated the performance of six ML algorithms, including adaptive boosting (AdaBoost), RF, GB, SVM, XGB, and ANN, in modeling the retrieval of three water quality parameters, namely Chl-a, turbidity, and TDS, in data-scarce conditions at Lake Tana, where traditional monitoring methods are prohibitively expensive. They assumed that ML models trained on limited data and satellite imagery could accurately predict water quality parameters. Their results showed that ensemble ML methods, particularly XGB and RF, performed best despite limited data. Using the best model, they produced monthly water quality maps to assess spatial and temporal variability in Chl-a, turbidity, and TDS distribution. The key findings are that XGB or AdaBoost are recommended for Chl-a retrieval, while RF or XGB can retrieve TDS and turbidity from Landsat 8 OLI imagery. These ML approaches can produce fast, highly accurate water-quality maps needed for source identification, trend monitoring, and understanding influencing factors. These outputs have valuable implications for researchers and policymakers designing strategies for sustainable water management in tropical lakes and similar environments.

Nafsin and Li [31] evaluated several ML algorithms for BOD prediction in the Buriganga River system in Bangladesh. They applied four standalone ML models, such as ANN, SVM, RF, and GB, and six new hybrid models, such as RF-SVM, ANN-SVM, GB-SVM, RF-ANN, GB-ANN, and RF-GB. Feature importance analysis by RF showed that important predictors for BOD were COD, TDS, conductivity, TS, TSS, and turbidity. Among the models, RF-SVM had the highest predictive accuracy with an R^2 of 0.908.

Uddin et al. [32] presented a framework for evaluating WQI model performance in classifying water quality with precision in Cork Harbor, Ireland. The authors used four ML classifier algorithms to identify the best algorithm for predicting water quality classes. The main conclusions of the survey are as follows: (1) seven WQI models are evaluated, including four widely used and three newly proposed. (2) The XGB algorithm and K-nearest neighbor (KNN) surpassed the rest in terms of correct WQC. (3) XGB satisfactorily classified most of the WQCs, saving the class "poor." (4) weighted quadratic mean (WQM)-WQI and unweighted RMS-WQI models turned out to be good enough for the correct assessment of coastal water quality status. (5) By both models, the water quality in Cork Harbour was classified as "good" and "fair" during the period under study. The results of this study are important for developing WQI models that are more accurate, transparent, and reliable in classifying water quality along coastlines. One important limitation of the present research was the absence of consideration of temporal variability regarding water quality indicators in Cork Harbor.

Nanjappachetty et al. [33] estimated WQI by using a novel approach coupled with advanced optimization algorithms and deep capsule crystal edge graph neural networks (DCCEGNN). They applied the hybrid crested porcupine Genghis Khan shark optimization (CPGKSO) algorithm to ensure careful feature selection, using only the most relevant water quality indicators. The authors also applied the Greylag Goose optimization (GGO) algorithm to further refine the neural network's weight parameters and improve prediction accuracy. The results showed that the proposed model presents an MSE of 6.7 with 99% accuracy in WQI prediction.

Jahan and Pradhanang [34] proposed an ANN model to predict high chloride concentrations resulting from road salt and deicers

in a suburban watershed. The research used three years of data (2016-2019) from six sites to study seasonal chloride changes and differences between agricultural zones and impervious surfaces. Chloride concentrations during winter runoff were considerably higher than in other seasons. The ANN model used hydrometeorological data, stormwater runoff quality, and network parameters as input variables. We used data from 42 storm events across all four seasons for training. Sensitivity analyses identified the input variables that controlled model performance. Using the backpropagation algorithm and cross-validation, the ANN achieved about 91% prediction accuracy, with 95% confidence intervals for its outputs. These results show that storm characteristics such as duration, intensity, and runoff volume significantly impact chloride concentrations, particularly during the winter months, due to road salt application. During the non-winter seasons, chloride detection was poor and most likely due to fertilizer applications in agricultural areas. The spatial distribution analysis showed a higher percentage of chloride clustering around impervious parking lots (about 70%), while less than 50% was in agricultural zones, suggesting that road salt is the major source. ANN modeling proved a powerful tool for predicting water quality parameters like chloride and is practical and efficient for environmental management.

Overall, AI models in surface water quality modeling have shown strong potential to capture spatial and temporal variability across diverse regions. While ensemble and hybrid models such as CATBoost, GPR, and LSTM-based combinations have consistently outperformed traditional approaches in predictive accuracy, concerns about overfitting, limited interpretability, and limited generalizability persist. Notably, studies often lack details on data preprocessing, feature selection, and validation against unseen datasets, which are critical for real-world deployment. Furthermore, the lack of explainable AI (XAI) integration limits transparency in model decisions, especially in policy-making contexts. Future research should incorporate standardized benchmarking, uncertainty quantification, and cross-regional validation to strengthen the practical utility and trustworthiness of AI-driven water quality monitoring frameworks. Fig. 2 demonstrates the overall framework of surface water quality determination.

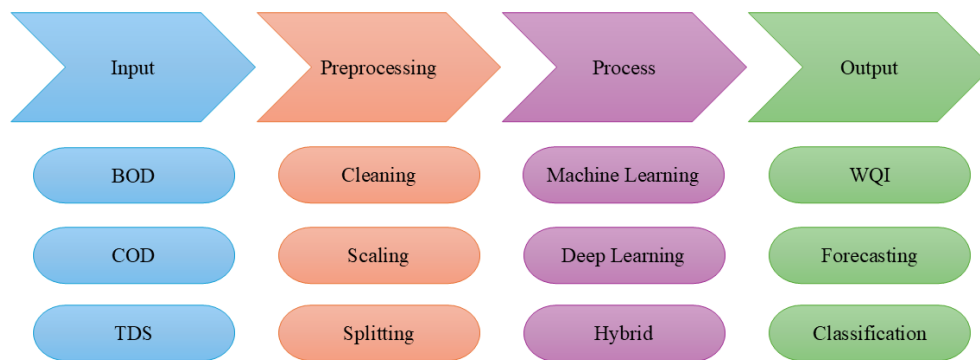


Fig. 2. Overall framework of surface water quality assessment.

Fig. 3 shows the distribution of AI models used to determine surface water quality. RF, SVM, XGB, and ANN are the most common models for determining surface water quality. LSTM, bagging-RF, ANFIS, SVR, MLR, RS-RF, and GB models rank next. Other models, including more than 30, account for 55% of all studies. Given that these models account for almost half of studies, they are considered more suitable for determining surface water quality. This may be because they can analyze large datasets with many features, where models such as XGB and RF are stronger, as well as the time-series forecasting ability of ANN and LSTM models.

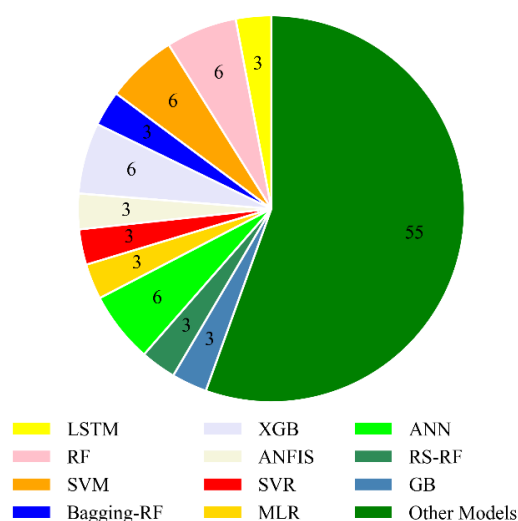


Fig. 3. Model distribution of surface water study by percent.

3.2. Groundwater

Goodarzi et al. [35] applied the world health organization (WHO) WQI and fuzzy hierarchical analysis process (FAHP)-WQI to evaluate the water quality of 96 wells in the Yazd-Ardakan plain and compared the results of both methods. In the WQI (WHO) results, 72 of the 96 wells were found to have good-quality water, and the rest were categorized as poor. In contrast, 31 wells showed good water quality when assessed with the FAHP-WQI method, 23 were poor, 22 were not usable, and 4 were excellent. In addition,

water quality classifications for agricultural purposes were also made using Wilcox and Schoeller's diagrams, respectively. From the Wilcox diagram, about 37.25% of the wells fell into the C3S2 and C3S1 classes, indicating low quality and suitability only for irrigation of specified crops in well-drained, large-textured fields. Schoeller's diagram divided drinking water quality into three classes: acceptable, bad, and inappropriate. Because calculating the WQI is time-consuming, this research estimated WQI using AI algorithms. Gene expression programming (GEP), M5P, and MARS were selected, and WQI calculations were carried out. In this study, models were evaluated using statistical indices such as Nash-Sutcliffe efficiency, RMSE, the agreement index, R, and MAE. These were then compared to identify the best model for WQI estimation. The GEP model data were relatively scattered, which is why its R was lower than that of the other models. The M5P and MARS model results are very similar. For the MARS model, during the training phase, the RMSE was 0.172, MAE 0.127, NSE 1, and R 1. During the test phase, the respective values were 0.212, 0.167, 1, 0.999, and 0.999. Although the MARS model had lower accuracy in the testing phase, it produced the best statistical indicator values among the three.

Kulisz et al. [36] carried out model studies using empirical data from 19 wells around a shale gas drilling rig in Poland. The developed ANN model used a two-layer feed-forward architecture with sigmoid hidden neurons and linear output neurons, fed by five input parameters: EC, pH, Ca, Mg, and K. Multiple regression results were used to optimize the input data. For the ANN model, an overall high R of 0.9992 and MSE of 0.0292, 0.2131, and 1.3244 for the training, testing, and validation datasets, respectively, were achieved. These were developed with dataset divisions of 70% training, 15% validation, and 15% testing using the NN fitting app and the Levenberg-Marquardt algorithm for training. The proposed ANN model provides a straightforward method for assessing groundwater pollution levels without the cumbersome calculations involved in conventional WQI methods.

Shiri et al. [37] applied eight soft computing techniques for modeling four critical groundwater quality parameters, namely EC, SAR, TDS, and SO₄, using data drawn from 90 observation wells in northwest Iran. Two modeling strategies were pursued: one predicted the targets based on other quality parameters, and the second solely based on geographical information (coordinates). The results show that all models simulated the target values appropriately, and the best among them was the coupled SVM-firefly algorithm (FFA). The models also perform well using only geographical inputs. One key takeaway is that the most appropriate k-fold testing strategy can deliver robust model performance, as obvious fluctuations were observed across different test stages. The results therefore suggest further studies using data from catchments under different climatic conditions and data availability to validate findings on comparative soft computing approaches.

Agrawal et al. [38] studied the estimation of drinking water quality using PSO and SVM. They collected groundwater samples from the Pindrawan tank command area and tested them against the developed model. They analyzed these samples for various water quality parameters, followed by WQI calculation. Some of the key inferences that can be made from the study are as follows: (1) The values of WQI indicated that 32.43% of water samples fall into the category of excellent quality, and 43.24% showed good quality. In contrast, 21.62% and 2.71% of water samples fell into the poor and very poor categories, respectively. The Raikheda pond area showed very poor quality due to high chromium levels, whereas parts of the villages of Deogaon, Dhansuli, Bangoli, Amlitalab, and Khauna showed poor quality. (2) Major cations and anions analysis revealed that all anions were within the acceptable limits except potassium, with 13% of the samples exceeding the limit. Future heavy metal pollution from mining activities is expected, as 48.6% of samples in this study exceeded chromium limits, potentially causing health problems such as auditory problems, blood diseases, high blood pressure, and even death. For the estimation and prediction of WQI values accurately, the study proposed ensemble ML algorithms. Specifically, an ensemble method with PSO combined with a Naive Bayes classifier (NBC) was the best algorithm, achieving 92.8% accuracy in predicting WQI indices. This will help estimate WQI and, through the user interface, save time and effort. Overall, the research demonstrates the benefits of ensemble ML techniques by combining outcomes from multiple algorithms to improve overall accuracy. The developed algorithm can be applied globally to estimate water quality using a user interface. This study used a synthetic dataset developed with PSO; more real data would improve accuracy. The authors proposed using larger datasets, which could be obtained from the field, but this is cumbersome due to sampling and laboratory analysis. In addition, the algorithm could be further improved by fine-tuning based on performance studies using different input parameters.

Mattas et al. [39] focused on predicting irrigation water salinity, represented by EC, in the aquifers of the Gallikos River basin, a catchment area of intensive agricultural activities and small-scale enterprises vulnerable to pollution. In this framework, we developed ANN and MLR predictive models, leveraging their simplicity, satisfactory performance, and low input-data requirements. We used data from 233 samples collected over three years (2004–2006) to calibrate, validate, and evaluate the models. ANN and MLR models both showed strong predictive accuracy with the large dataset, as shown by high evaluation indices: $R^2 > 0.927$, modeling efficiency > 0.93 , and MAPE $< 14.5\%$. For smaller verification datasets comprising 18 samples, both models remained statistically reliable: $R^2 > 0.75$, modeling efficiency > 0.976 , and MAPE $< 20\%$, although MLR performed slightly better. We selected input parameters for both models using factor analysis to improve accuracy and reliability. These results highlight the practical relevance of ANN and MLR models for cost-effective, efficient water quality parameter forecasting, such as EC, and reduce the need for expensive, time-consuming in situ measurements. The present study provided a methodological framework applicable to other regions facing similar environmental challenges, offering actionable insights for parameter selection and model evaluation in water resource management.

Singha et al. [40] proposed a DL algorithm. They compared it with three other traditional ML models, such as RF, XGB, and ANN, for estimating the groundwater quality index (EWQI) in India. These metrics were then used to assess the performance and prediction capability of the models, and the results showed that the deep learning model performed best with the highest validation accuracy: MSE = 1.537, MAE = 1.360, RMSE = 1.254, RSR = 0.034, NSE = 0.987, MAPE = 0.959, and $R^2 = 0.996$. In comparison,

XGB performed better than both the RF and ANN models. Based on validation R^2 values, the performance ranking was $DL > XGB > ANN > RF$, with values of 0.996, 0.927, 0.917, and 0.886, respectively. These results demonstrate the DL model's efficiency in selecting the most influential input variables to compute EWQI. Therefore, the proposed DL model is suitable for the study area to evaluate the quality of the groundwater and can be applied in different water studies for managing the quality prediction parameters, including BOD, DO, and COD, and mainly for the removal of pollutants and toxic trace elements such as Pb, Cd, and Mn from surface water and groundwater. Here, the study is limited to a single monsoon dataset. If the dataset included several seasons, the results could be more informative about groundwater quality.

Zamili et al. [41] estimated the WQI of the Yesilirmak River in Çorum, Turkey, in the monitoring period from 1995 to 2014. Eleven physicochemical parameters were subjected to computation for WQI. Results ranged between good and poor; it was once found unsuitable for drinking in June 2000. This research also applied several new hybrid models in which pre-processing data and the ANN approach were optimized by different metaheuristic methods, namely coefficient-based PSO and chaotic gravitational search algorithm (CPSOCSA), marine predator's optimization algorithm (MPA), and PSO for WQI prediction. Furthermore, to improve model precision, in addition to WQ parameters, some climate factors, mainly rainfall, were considered predictors. Two key points emerged from the results of this study. First, the pre-processing methodologies improved the dataset and provided an appropriate set of predictors. Second, the CPSOCSA-ANN model outperformed the other models, including MPA-ANN and PSO-ANN, with $R^2 = 0.965$, MAE = 0.01627, and RMSE = 0.0187.

Sinha et al. [42] investigated the suitability of ANNs for modeling WQI in the region affected by industrial wastewater contamination in Jodhpur district, India. They assumed that water quality, controlled by multiple parameters such as pH, TDS, and TS, could be described by a single composite index that captures the overall impact of contamination. ANN modeling supported this hypothesis by reliably predicting the relationships among individual water quality parameters and overall WQI. The results presented a new stochastic WQI protocol applied to characterize water quality in Jodhpur, India, and concluded that potable water quality was generally good (~60% or above) despite industrial contamination. The results proved the efficiency of ANNs in modeling complex nonlinear interactions among water quality parameters and predicting their impact on potability. Although the findings agree with previous studies, the authors acknowledge limitations such as the lack of alternative algorithms and architectural comparisons.

El Bilali et al. [43] conducted a study to verify whether ML models efficiently predict irrigation water quality (IWQ) parameters using physical variables as input data, and to assess the sensitivity of ML models to these inputs. The study area was the Berrechid aquifer in north-west Morocco, between the cities of Settat and Casablanca. We developed four models to predict TDS, potential salinity (PS), SAR, exchangeable sodium percentage (ESP), magnesium adsorption ratio (MAR), and residual sodium carbonate (RSC) from their physical features using the AdaBoost, RF, ANN, and SVR algorithms. The results showed that the AdaBoost model was the best for all parameters, while RF was the second most successful, predicting five parameters except MAR. It was observed that the ANN and SVR models could predict the four fitness parameters TDS, PS, SAR, and RSC, and three fitness parameters, PS, SAR, and RSC, respectively. Moreover, all ensembling models (AdaBoost and RF) performed better than the conventional ANN and SVR models in all validation tests. ANN and SVR generalized better than AdaBoost and RF. In addition, Sensitivity analysis showed that the models are relatively insensitive to input variables across the range of predicted parameters. In general, ML models can efficiently forecast IWQ parameters and provide real-time, low-cost monitoring of irrigation water quality.

Overall, AI techniques in groundwater quality assessment have shown considerable promise, particularly in data-scarce and geochemically complex regions. Studies reviewed in this section have shown that hybrid models such as CNN-LSTM and ensemble-based approaches like MARS and M5P can achieve high predictive accuracy across diverse groundwater settings. These models can effectively handle nonlinearity and temporal variability in hydrochemical parameters. However, several limitations persist. Many studies do not address missing data, measurement uncertainty, and groundwater-specific features such as aquifer heterogeneity or seasonal recharge impacts. Furthermore, while ANNs and DL architectures achieve strong statistical performance, their limited interpretability hinders transparency for policy-level decision-making. Future research should prioritize integrating explainable AI (XAI) methods, robust validation across hydrogeological zones, and improved uncertainty treatment through probabilistic frameworks. Moreover, coupling AI models with GIS-based spatial tools could enhance spatiotemporal monitoring and support the design of sustainable groundwater management strategies [44–51]. Fig. 4 demonstrates the overall framework of groundwater quality determination.

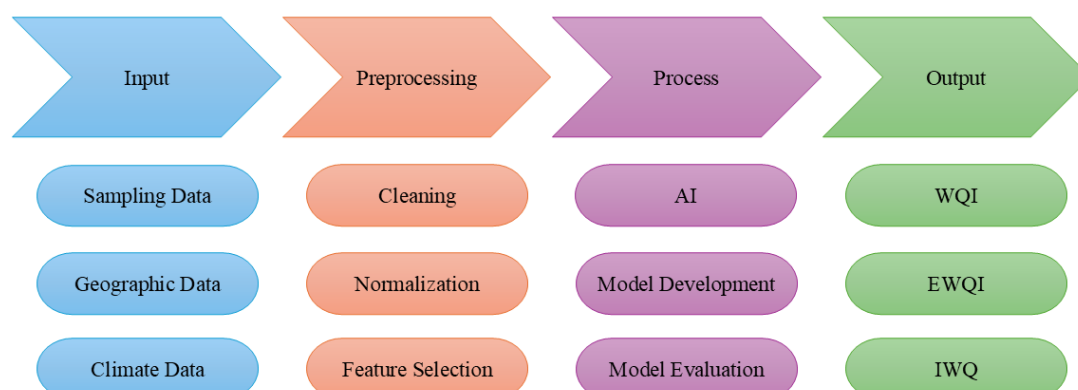
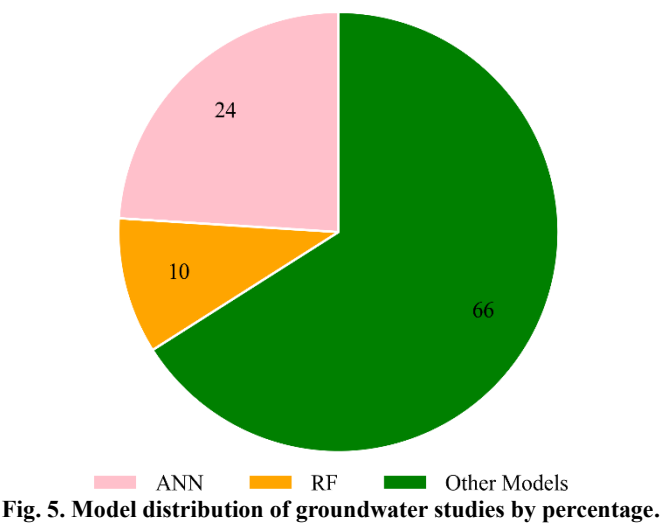


Fig. 4. Overall framework of groundwater quality assessment.

Fig. 5 shows the distribution of AI models used to determine groundwater quality. ANN and RF account for 24% and 10% of studies, respectively, and other models (more than 12) account for 66%. As mentioned, these models are preferred for groundwater quality because they effectively solve nonlinear relationships, handle missing data, and also provide high accuracy.



4. Comparative analysis and discussion

In this review, many AI methods have been used to determine water quality and have been published in recent years. The results indicate that different AI models have been successfully applied to predict water features across different geographical regions. Each model has its strengths and weaknesses. Fig. 6 provides a methodology with advantages and disadvantages. Table 2 illustrates the classification of AI by family.

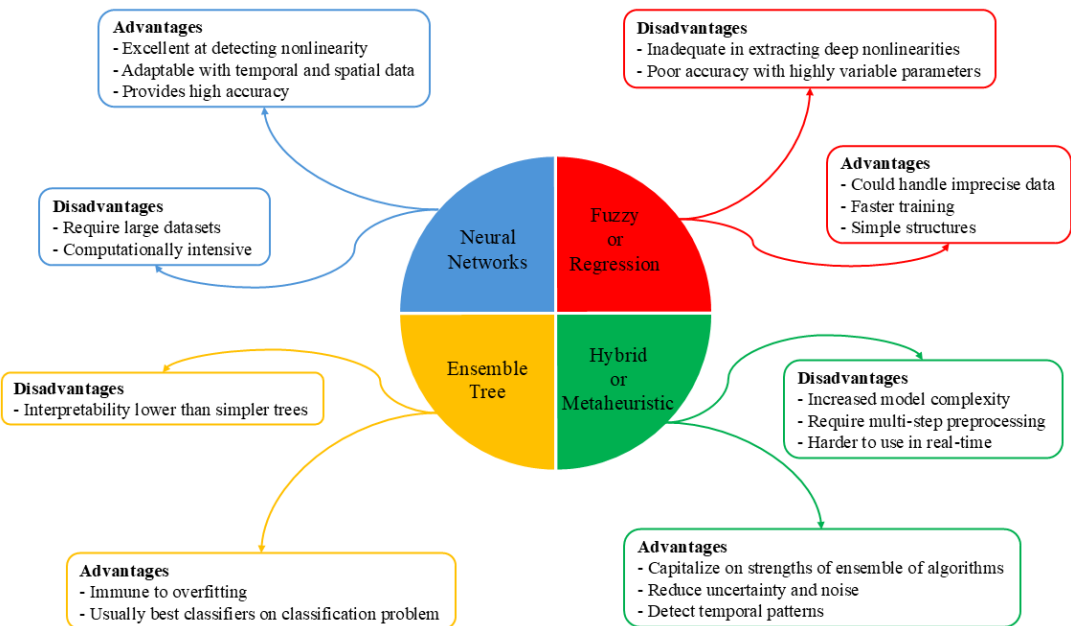


Fig. 6. Advantages and disadvantages of each methodology.

Table 2. Methodological families.

Method family	Artificial intelligence
Neural networks	ANN, MLP, CNN, LSTM, DL, CNN-LSTM, BPNN, MLPNN, RBFNN, ANN-WT-LSTM, DCCEGNN, NAR
Ensemble tree-based	RF, XGB, CatBoost, AdaBoost, GB, ETR, DT, RT, REPT, EN, bagging-RF, bagging-M5P, CVPS-REPT, RS-RF, RS-M5P, RC-RF, RC-M5P, AR-RF, AR-M5P
Hybrid/metaheuristic	CEEMDAN-XGB, CEEMDAN-RF, PSO, EFO, FFA, WCA, SCA, CPSOCGSA, MPA, CPGKSO, GGO, teaching-learning-based optimization
Fuzzy/regression-based	ANFIS, MLR, GEP, MARS, GPR, SLR, LR, SVM, KNN, SVR, DTR

As the tables and review show, no universal AI model performs best for all water quality parameters or under all conditions. Each algorithm is sensitive to the dataset's nature, the degree of nonlinearity, and the number of inputs. Therefore, hybrid and ensemble approaches have gained popularity in recent years, aiming to combine the strengths of multiple algorithms. Furthermore, studies show that preprocessing techniques such as decomposition (e.g., wavelet, CEEMDAN) and optimization (e.g., PSO, GWO) can significantly enhance model accuracy by reducing noise and selecting the most relevant features. These steps are essential in improving the overall modeling pipeline, not just the choice of algorithm.

Another general observation is the wide geographical and environmental diversity of study areas—from canals and lakes in South Asia to marshes in Iraq and groundwater aquifers in Europe. While surface water dominates the literature, AI use in groundwater quality modeling is also growing, particularly in regions where traditional monitoring is costly or logistically difficult. However, few studies address uncertainty quantification, real-time adaptability, or model explainability, limiting operational deployment in decision support systems. Future research should aim to address these gaps by integrating explainable AI methods, uncertainty analysis, and dynamic updating from sensor-based systems.

4.1. Critical factors affecting AI model performance

AI model performance in water quality prediction depends not only on algorithm structure but also on dataset characteristics, environmental conditions, input variables, and hydrological complexity. The reviewed studies demonstrate that hybrid and ensemble models generally outperform single algorithms because they combine different learning mechanisms and reduce uncertainty. For example, RF-SVM achieved superior BOD prediction performance by integrating feature selection capability with nonlinear classification strength, while CNN-LSTM models improved prediction by simultaneously capturing spatial and temporal patterns. Dataset size and quality are among the most important factors affecting model reliability. Studies using long-term monitoring datasets, such as the 36-year river salinity dataset analyzed by Melesse et al. [29], achieved higher generalization than studies based on limited seasonal datasets. However, large datasets alone do not guarantee better performance; preprocessing, noise reduction, feature selection, and appropriate validation procedures significantly influence the final accuracy. Hydrological characteristics also control model suitability. Surface water systems generally involve strong temporal variability caused by rainfall, runoff, and pollution events, making recurrent models such as LSTM more suitable for time-series prediction. Conversely, groundwater quality modeling is influenced by aquifer heterogeneity, recharge conditions, and geochemical interactions; therefore, models that handle nonlinear relationships, such as ANN, ANFIS, and hybrid approaches, have performed better.

The number and type of input parameters also affect prediction accuracy. Several studies demonstrated that feature selection can reduce unnecessary variables without reducing model performance. For instance, some WQI models achieved reliable predictions using only key parameters such as BOD, turbidity, and phosphate, rather than many measurements. Climatic conditions further influence AI model applicability. Models developed in humid, tropical, arid, or highly seasonal environments may not directly transfer to other regions because hydrological processes, pollutant transport mechanisms, and seasonal variability differ significantly. Therefore, regional validation and uncertainty analysis remain essential for developing transferable AI-based water quality prediction systems.

4.2. Bibliometric analysis of reviewed studies

To provide an additional perspective on the evolution and research trends of AI-based water quality prediction, we conducted a bibliometric analysis based on the 35 studies selected through the PRISMA procedure. The analysis focused on publication trends and keyword co-occurrence patterns to identify the field's development trajectory and dominant research themes.

Fig. 7 presents the annual distribution of the reviewed publications. The results indicate a continuous research interest in AI applications for water quality assessment, with a noticeable increase in publications after 2020. This growth corresponds with the rapid development of machine learning, deep learning, and hybrid modeling approaches in environmental and hydrological applications. The growing number of studies highlights the shift from conventional statistical approaches to data-driven prediction frameworks.

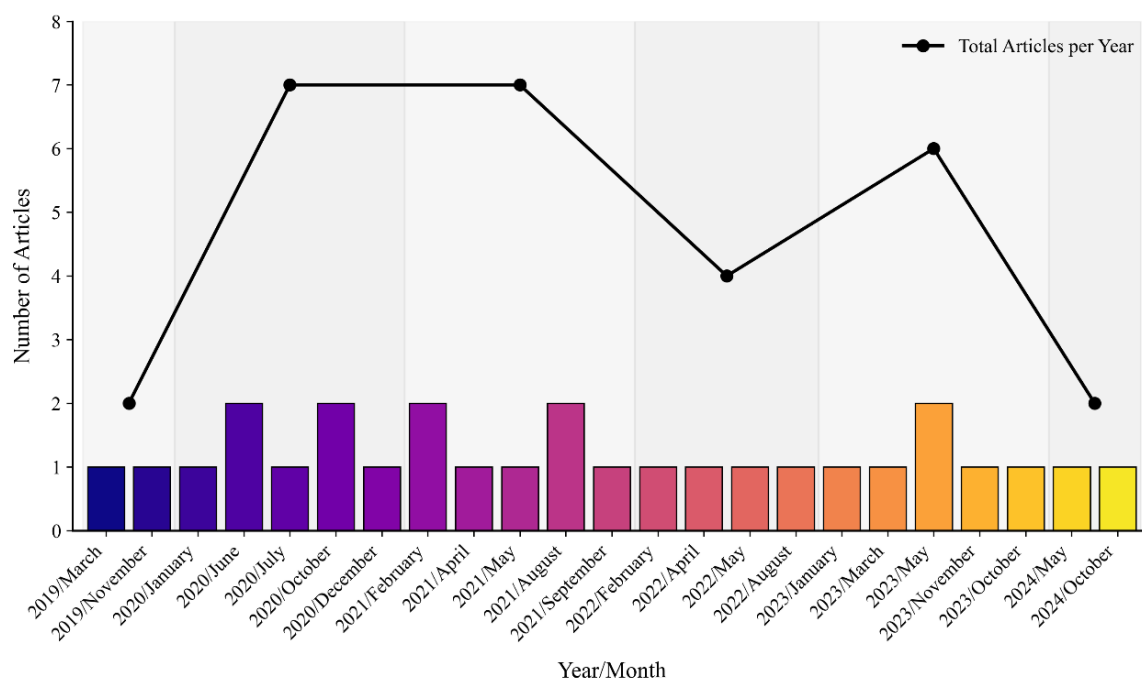


Fig. 7. Annual publication trend of AI-based water quality prediction studies included in the review.

We performed keyword co-occurrence analysis to identify major research themes and relationships among frequently used terms. As shown in Fig. 8, "water quality index," "machine learning," "artificial neural network," and "water quality" are central research themes with strong connections to various modeling approaches, prediction objectives, and environmental applications. The network shows the field's evolution from individual machine learning applications to integrated frameworks involving hybrid algorithms, ensemble learning, uncertainty analysis, and advanced optimization techniques.

The observed clusters indicate several major research directions. One cluster includes AI modeling approaches such as artificial neural networks, random forests, support vector machines, and deep learning methods. Another cluster focuses on water quality indicators and environmental applications, including groundwater quality, electrical conductivity, total dissolved solids, and water quality indices. A further emerging theme relates to hybrid modeling, feature selection, optimization algorithms, and explainable prediction frameworks.

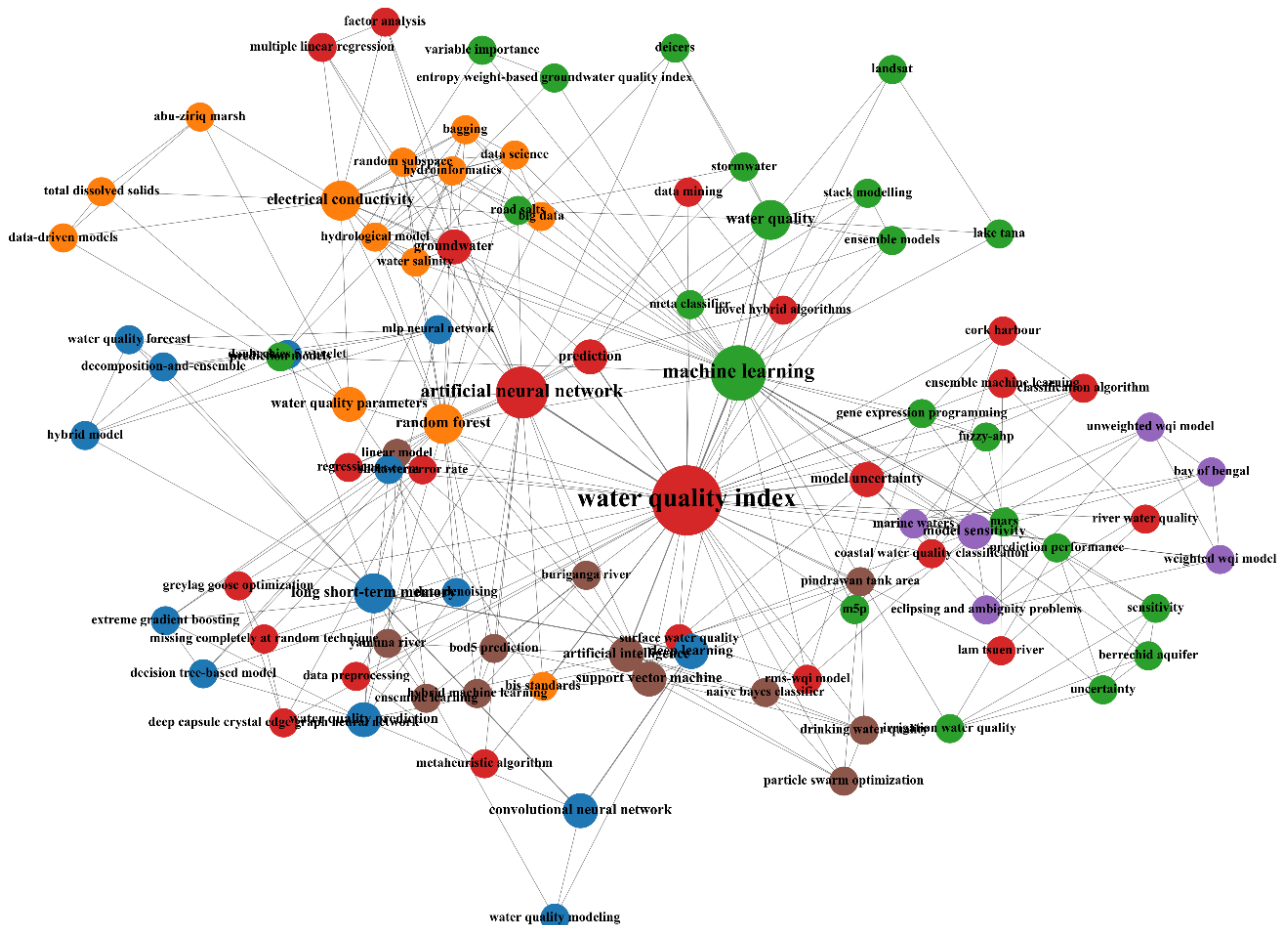


Fig. 8. Geographical distribution of reviewed studies.

Because the number of included studies is relatively small, we did not perform advanced citation-based network analyses such as institutional collaboration and co-citation mapping. Fig. 9 shows the geographical distribution of reviewed studies. Moreover, Fig. 10 demonstrates the distribution of reviewed studies by journal.

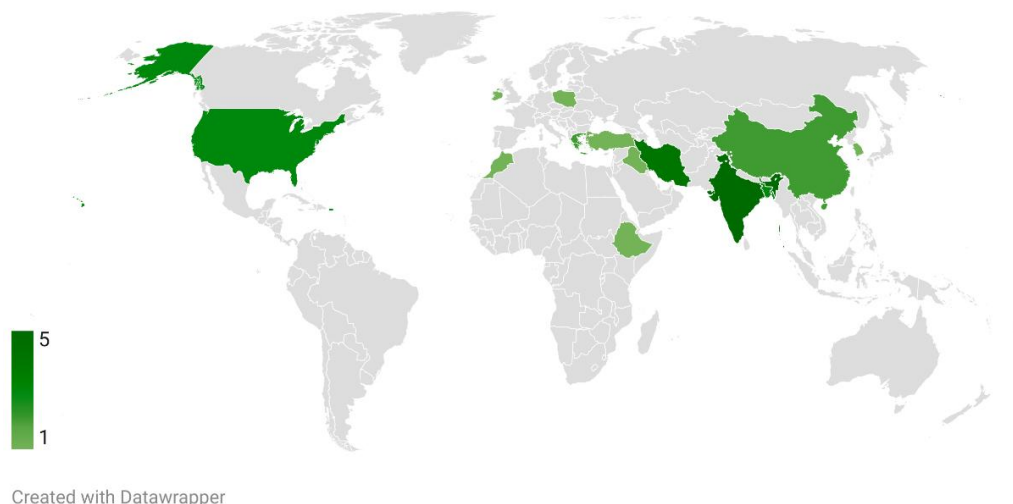


Fig. 9. Geographical distribution of reviewed studies.

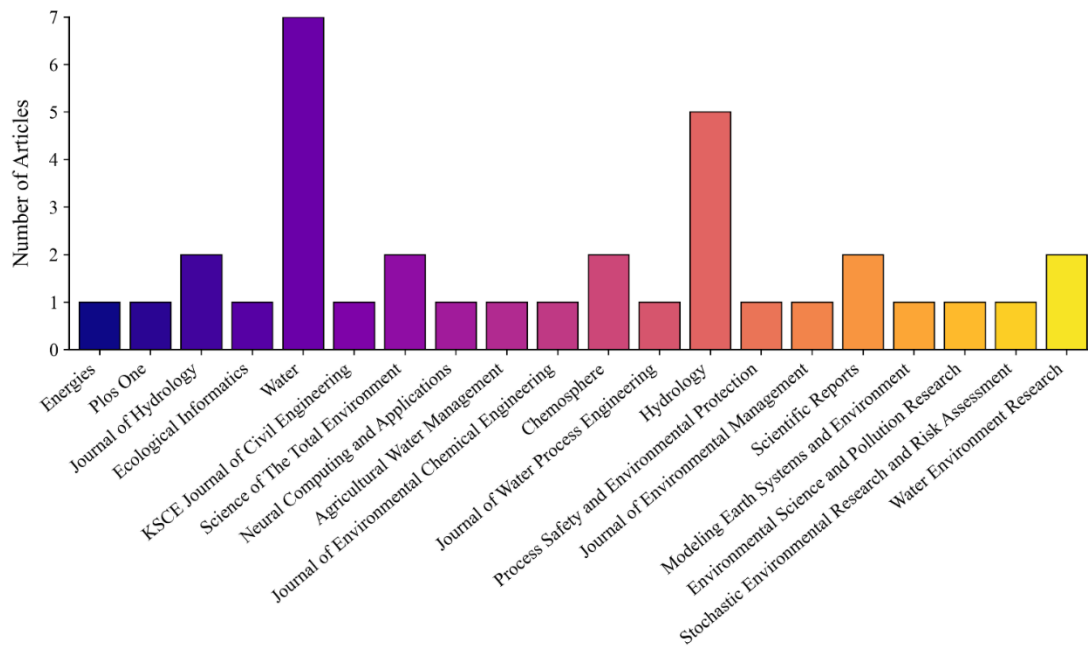


Fig. 10. Distribution of reviewed studies by journal.

4.3. Study distribution

This review showed that about 65% discussed surface water and 35% discussed groundwater. Methodologically, about 68% used neural networks, 46% used tree ensemble models, 29% used hybrid or metaheuristic approaches, and 50% used fuzzy logic or regression methods, with some overlap because each study used different methods. Fig. 11 indicates this statistic.

The distribution of groundwater studies and surface water studies is as follows: neural network models were used in 68% of surface water and 67% of groundwater studies; tree ensemble models were used in 53% of surface water and 33% of groundwater studies; hybrid or metaheuristic approach models were used in 26% of surface water and 33% of groundwater studies; fuzzy logic or regression methods were used in 47% of surface water and 56% of groundwater studies. Fig. 12 shows the distribution of method families in surface water and groundwater studies. Table 3 also lists each study and its method family.

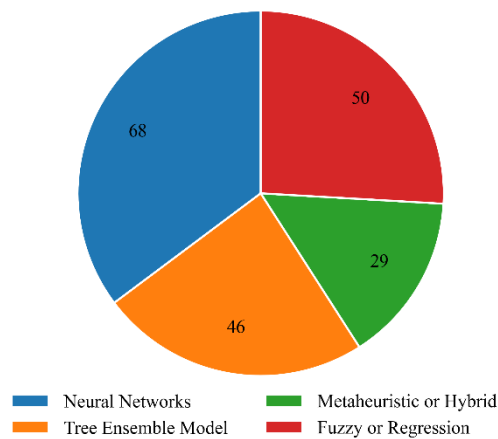


Fig. 11. Study distribution by methodological families by percent.

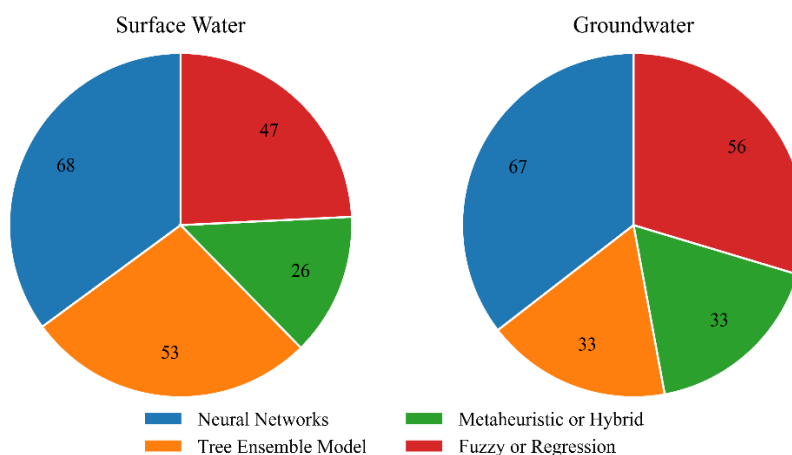


Fig. 12. Method family in surface water and groundwater by percent.

Table 3. Research distribution by each method family.

Method family	Surface water	Groundwater
Neural networks (ANN, MLP, CNN, LSTM, DL, CNN-LSTM, BPNN, MLPNN, RBFNN, ANN-WT-LSTM, DCCEGNN, NAR)	Wang et al. [16]	
	Nasir et al. [17]	
	Yang [20]	
	Abba et al. [21]	
	Wu and Wang [22]	Kulisz et al. [36]
	Lu and Ma [24]	Mattas et al. [39]
	Al-Mukhtar and Al-Yaseen [25]	Singha et al. [40]
	Barzegar et al. [26]	Zamili et al. [41]
	Baek et al. [27]	Sinha et al. [42]
	Leggesse et al. [30]	El Bilali et al. [43]
	Nafsin and Li [31]	
	Nanjappachetty et al. [33]	
	Jahan and Pradhanang [34]	
Ensemble tree-based (RF, XGB, CatBoost, AdaBoost, GB, ETR, DT, RT, REPT, EN, bagging-RF, bagging-M5P, CVPS-REPT, RS-RF, RS-M5P, RC-RF, RC-M5P, AR-RF, AR-M5P)	Nasir et al. [17]	
	Bui et al. [18]	
	Sajib et al. [19]	
	Asadollah et al. [23]	
	Lu and Ma [24]	Goodarzi et al. [35]
	Uddin et al. [28] - Bay of Bengal	Singha et al. [40]
	Melesse et al. [29]	El Bilali et al. [43]
	Leggesse et al. [30]	
	Nafsin and Li [31]	
	Uddin et al. [32] - Ireland	
Hybrid/metaheuristic (CEEMDAN-XGB, CEEMDAN-RF, PSO, EFO, FFA, WCA, SCA, CPSOCGSA, MPA, CPGKSO, GGO, teaching-learning-based optimization)	Nasir et al. [17]	
	Yang [20]	Shiri et al. [37]
	Lu and Ma [24]	Agrawal et al. [38]
	Nafsin and Li [31]	Zamili et al. [41]
	Nanjappachetty et al. [33]	
Fuzzy/regression-based (ANFIS, MLR, GEP, MARS, GPR, SLR, LR, SVM, KNN, SVR, DTR)	Nasir et al. [17]	
	Sajib et al. [19]	
	Abba et al. [21]	Goodarzi et al. [35]
	Asadollah et al. [23]	Shiri et al. [37]
	Lu and Ma [24]	Agrawal et al. [38]
	Al-Mukhtar and Al-Yaseen [25]	Mattas et al. [39]
	Leggesse et al. [30]	El Bilali et al. [43]
	Nafsin and Li [31]	
	Uddin et al. [32] - Ireland	

Table 3 shows that, by priority, popular methods for surface water are neural networks, ensemble tree-based methods, fuzzy/regression-based methods, and hybrid/metaheuristic methods. For groundwater, the trend is neural networks, fuzzy/regression, and metaheuristic/tree-based methods, respectively. This popularity demonstrates the power of neural networks in determining water quality in both surface water and groundwater because they can model nonlinear relationships. This classification is intended only to provide a general idea of which AI family is more commonly applied to computing surface water or groundwater parameters, not a specific water quality parameter. This is because, when computing a specific parameter such as DO or TDS, a different type of AI can be more efficient. This is because the degree of linearity or non-linearity in each water parameter's variation differs. Since each AI model specializes in a certain kind of computation, it provides more accurate results for that parameter.

4.4. Suitable AI methods by water quality parameter

- On WQI, different ML models have estimated surface and groundwater with high accuracy. In surface water, GPR had the lowest uncertainty ($< 2\%$) and the highest R^2 in inland canal studies, such as the Tongi canal, because it effectively estimates predictive uncertainty in ecological data. Additionally, the CEEMDAN-XGB model performed better at identifying both short-term and long-term trends by denoising and then boosting, with MAPEs as low as 0.27% for pH and 14.94% for turbidity. MARS and M5P hybrid models in groundwater analysis achieved near-perfect training and testing correlation ($R \approx 1.00$) using linear splines to model sudden changes in well chemistry. CPSOCGSA-ANN also outperformed other metaheuristic approaches, achieving $R^2 = 0.965$ when optimized for long-term river monitoring.
- RF-SVM hybrid ($R^2 = 0.908$) for Bangladesh's BOD was optimally combined using RF's strength and SVM's margin maximization of nonlinear relationships.
- In DO determination, the EFO-MLPNN hybrid required fewer iterations and agents but achieved improved RMSE and NSE by means of efficient network weight search space exploration. LSTM-hybrids based on ANN-WT-LSTM also improved short-term DO trend predictions using wavelet decomposition to eliminate noise.
- In pH and turbidity, CEEMDAN-XGB reduced noise effects to reach MAPEs $< 1\%$ in pH prediction and dominated extreme turbidity oscillations.
- In EC and TDS columns, ANFIS applied fuzzy logic to reduce input uncertainties in marsh and marshland conditions to surpass MLR and ANN in predicting TDS and EC.
- In Chl-a, TN, and TP assessment, CNN models retained high-resolution spatial patterns in spectral data, while hybrid CNN-

LSTM added temporal dynamics to reduce RMAE by up to 6.5% on Chl-a. Additionally, XGB and RF also performed effectively for Chl-a prediction.

4.5. Structured overview of AI applications in water quality modeling

To support this analysis, Table 4 categorizes each reviewed study by AI model, key advantages and limitations, water resource type, application area, accuracy level, usage location, and suitability.

Table 4. Summary of AI-based water quality prediction models with comparative advantages, limitations, and suitability across various hydrological contexts.

Artificial intelligence (AI) model	Advantages	Disadvantages	Water resource	Application area	Accuracy	Usage location	Suitability	Ref. No.
LSTM	Predicts industrial pollution impact	Limited to a single-point prediction	Surface water	River water quality management	High accuracy in predicting water quality trends	Industrial river basins, China	Pollution management	Wang et al. [16]
CATBoost, RF, MLP, SVM, DT	High accuracy, improved predictions	Model complexity	Surface water	WQC	CATBoost: 94.51%, Stacking: 100%	India	Complex analyses	Nasir et al. [17]
BA-RT, bagging-RF, CVPS-REPT	Better performance than standalone models	Poor performance with extreme WQI values	Surface water	WQI prediction	BA-RT: Highest performance	Mazandaran and Semnan provinces in Iran	Groundwater management	Bui et al. [18]
GPR, XGB, SVM, NN	Best performance for WQI	Limited to a single water body	Surface water	WQI prediction	Best in RMSE, MAE, R ²	Tongi canal, Bangladesh	Urban water quality analysis	Sajib et al. [19]
EFO, WCA-MLPNN, SCA	High accuracy, faster computation	Requires many iterations for EFO	Surface water	DO prediction	Superior in RMSE, RP, NSE	Klamath River, Oregon, USA	DO prediction	Yang [20]
BPNN, ANFIS, SVR, MLR	Improved accuracy with ensemble models	Performance varies by location	Surface water	WQI prediction	ANFIS: 30-53% improvement in absolute error	Yamuna River, India	WQI prediction in rivers	Abba et al. [21]
ANN-WT-LSTM, ANN-LSTM	High accuracy, long-term, and short-term prediction	Limited spatial variation predictions	Surface water	Water quality prediction	Outperforms ANN-LSTM	Jinjiang River, China	Suitable for early warning systems	Wu and Wang [22]
ETR, SVR	Higher accuracy than SVR and DTR	Sensitive to input variables	Surface water	WQI prediction	Best with three inputs	Lam Tsuen River, Hong Kong	Real-time WQI monitoring	Asadollah et al. [23]
CEEMDAN-XGB, CEEMDAN-RF, PSO-SVM, RBFNN	High accuracy, model stability	Limited to the water quality of the specific site (Gales Creek)	Surface water	Water quality prediction	MAPE: 0.69%-14.94%	Gales Creek, Oregon, the USA	Water quality monitoring	Lu and Ma [24]
ANFIS, ANN, MLR	Provides robust predictive frameworks for TDS and EC	Higher complexity of ANFIS structure	Surface water	Water quality (TDS, EC) in marshes	Best for ANFIS outperforms others	Abu-Ziriq Marsh, Iraq	Designing and managing water resource systems	Al-Mukhtar and Al-Yaseen [25]
CNN-LSTM, DL	Combines strengths of CNN and LSTM	Limited in high-level performance	Surface water	DO and Chl-a prediction	RMAE: 0.24% improvement in DO	Small Prespa Lake, Greece	Multi-level prediction	Barzegar et al. [26]
CNN, LSTM	Accurate temporal modeling	Limited to three pollutants	Surface water	Water level and pollutant simulation	NSE > 0.75	Nakdong River basin, South Korea	Pollutant and water level simulation	Baek et al. [27]
XGB	Efficient marine water assessment	Limited spatial coverage	Surface water	Coastal water quality assessment	RMSE, MAE < 1%, no ambiguity	Bay of Bengal	Marine water quality monitoring	Uddin et al. [28]
M5P, RF, bagging-M5P, bagging-RF, RS-M5P, RS-RF, RC-M5P, RC-RF, AR-M5P, AR-RF	Better than standalone models	Inconsistent performance when hybridized	Surface water	River salinity prediction	R = 0.99	Babol-Rood River, Iran	River salinity prediction	Melesse et al. [29]
AdaBoost, RF, GB, SVM, XGB, ANN	High performance in data-scarce environments	Limited to satellite imagery	Surface water	Modeling Chl-a, turbidity, and TDS	High (XGB and RF recommended)	Lake Tana, Ethiopia	Quick, reasonably accurate water quality monitoring in tropical lakes	Leggesse et al. [30]

ANN, SVM, RF, GB, RF-SVM, ANN-SVM, GB-SVM, RF-ANN, GB-ANN, RF-GB	High prediction accuracy, cost-effective, systematic data management	Requires significant computational resources	Surface water	BOD prediction	$R^2 = 0.908$	Buriganga River, Bangladesh	Developing countries with pollution control needs	Nafsin and Li [31]
XGB, KNN	High accuracy in classification	Poor performance in the "poor" category	Surface water	Water quality classification	XGB is best in most classes	Cork Harbor, Ireland	Coastal water quality	Uddin et al. [32]
DCCEGNN, CPGKSO, GGO	Extremely high accuracy, robust optimization, precise feature selection	Complex model requiring advanced expertise	Surface water	WQI prediction	99% (MSE = 6.7)	Global freshwater monitoring	Environmental authorities managing river quality	Nanjappachetty et al. [33]
ANN	High prediction accuracy	Requires significant hydrometeorological data	Surface water	Chloride prediction in watersheds	Prediction accuracy: 91%	Suburban watershed, the USA	Evaluating the impact of road salt and seasonal runoff on chloride levels	Jahan and Pradhanang [34]
GEP, MSP, MARS	High accuracy during training	Lower accuracy in the testing phase	Groundwater	Groundwater quality prediction	MARS best in statistical indicators	Yazd-Ardakan Plain, Iran	Groundwater WQI estimation	Goodarzi et al. [35]
ANN	High R	Limited dataset	Groundwater	Groundwater pollution assessment	$R = 0.9992$	Shale gas site, Poland	Groundwater pollution monitoring	Kulisz et al. [36]
SVM-FFA	Accurate predictions with geographical inputs	Fluctuations in test stages	Groundwater	Groundwater quality modeling	Best with SVM-FFA	Northwest Iran	Catchment area quality analysis	Shiri et al. [37]
PSO, SVM	High prediction accuracy	Limited real dataset	Groundwater	WQI estimation	Accuracy: 92.8%	Pindrawan, India	Drinking water quality estimation	Agrawal et al. [38]
ANN, MLR	Cost-effective and time-saving	Results depend on data availability	Groundwater	Irrigation water salinity prediction	$R^2 > 0.927$	Gallikos River basin, Greece	Forecasting water salinity efficiently with minimal resources	Mattas et al. [39]
DL, XGB, RF, ANN	Highest validation accuracy	Limited dataset (monsoon season only)	Groundwater	Groundwater quality prediction	DL accuracy: $R^2=0.996$	India	Pollutant and toxic element prediction	Singha et al. [40]
CPSOCGSA-ANN, MPA-ANN, PSO-ANN	Enhanced dataset preprocessing	Limited consideration of climate factors	Groundwater	WQI prediction	CPSOCGSA-ANN: $R^2 = 0.965$	Yesilirmak River, Turkey	WQI prediction using metaheuristics	Zamili et al. [41]
ANN	Introduced a stochastic WQI protocol	Limited exploration of the alternative model	Groundwater	Industrial wastewater regions	Satisfactory for WQI	Jodhpur, India	Assessing water potability despite contamination from industrial sources	Sinha et al. [42]
AdaBoost, RF, ANN, SVR	Efficient in IWQ prediction	ANN and SVR generalize less effectively	Groundwater	IWQ prediction	AdaBoost outperforms others	Irrigation systems in Morocco	Real-time IWQ monitoring	El Bilali et al. [43]

By analyzing Table 4, it can be concluded that:

- LSTM model: From the results, this model appears to provide accuracy in predicting industrial water pollution impacts, particularly for river water quality management. This makes it well suited for managing or controlling pollution in industrial river basins, since it can precisely predict water-quality trends.
- CATBoost RF stacking ensemble: Despite the complexity of this ensemble method, it has reached an incredible accuracy level regarding WQC tasks. For example, this stacking model reached 100% accuracy in India. Therefore, this model may prove extremely effective for complex water-quality analysis.
- Hybrid models - BA-RT: Bagging-RF - hybrid algorithms are better independent models, especially useful for WQI predictions in areas where routine monitoring is scarce. However, scores tend to drop when dealing with extreme WQI values, which is challenging.
- GPR: The model performed best in predicting WQI for urban areas, as in the case of the Tongi canal in Bangladesh, with the highest accuracy for different statistical metrics such as RMSE, MAE, and R^2 . This model has proved highly reliable for assessing urban water quality.

- Results showed that AI and ML models have successfully predicted water quality and related environmental parameters beyond expectations. However, each model had its respective limitations. For instance, LSTM models were bound to single-point prediction, making them inappropriate for large-scale or multi-point water monitoring systems. On the other hand, hybrid models such as BA-RT may perform well in areas with no regular monitoring; however, they perform poorly for extreme WQI values.
- Advanced models such as CATBoost, RF, and stacking ensemble perform much better in the classification task, especially regarding water quality studies in such complicated environments. Yet this advantage often comes with increased computational complexity, which can raise costs and resource demands.
- Generally speaking, the region, characteristics of a dataset, and set goals will determine which model is most appropriate. For instance, CNN-LSTM models are well-suited for multi-level predictions. In contrast, models such as XGB and RMS-WQI are sufficiently powerful for accurate assessment of marine water quality in coastal regions.
- The analysis pinpointed that model selection should be carried out concerning unique characteristics pertinent to the task of monitoring water quality.

In summary, model selection for water quality monitoring should always consider the unique dynamics of the target parameter, the spatial-temporal scale, and the availability of data and computational resources.

4.6. Toward a unified analytical framework

To enhance water quality modeling, we propose a hybrid explainable AI-spatiotemporal decomposition (XAI-STD) framework that integrates data decomposition, feature selection, model ensemble, interpretability, and real-time adaptability. The framework begins with data preprocessing and multiscale decomposition, utilizing methods such as WT or CEEMDAN to remove noise and extract intrinsic signal patterns across temporal and spatial resolutions.

It then selects the most significant features using metaheuristic optimizers such as PSO or GWO to optimize the pool of physicochemical variables and remote sensing indicators chosen for the model's input stage. This feature selection optimizes the input variable set. In the modeling stage, a hybrid model approach is taken, using an ensemble of three algorithmic approaches with different specialties:

An uncertainty estimator probabilistic regressor model (e.g., Gaussian process regression, GPR), which models both the predictive value and the confidence intervals for this prediction; an ensemble tree-based machine learning model (e.g., XGBoost) with nonlinear boosting capabilities; and a recurrent neural network (e.g., LSTM) to account for potential temporal dependencies or temporal data patterns.

Finally, real-time responsiveness and adaptability are sought through learning algorithms with incremental or adaptive retraining methods, using a continuous data stream from IoT sensors to keep model performance up to date. In summary, the proposed XAI-STD modeling framework is expected to be able to:

Represent the nonlinear spatiotemporal dependencies of water quality variables in an open water reservoir, explain its predictions transparently and intuitively for all three input feature types, and remain adaptive and responsive for real-time updates in water quality monitoring.

This modeling framework is computationally intensive, requires domain and machine learning expertise, and involves implementation costs. Nonetheless, it may suit high-stakes water resource management and optimization use cases where a compromise between prediction accuracy, explainability, and adaptability is not possible.

5. Conclusions

In this literature review, we analyzed recent works that used artificial intelligence (AI) techniques for water quality prediction. The papers selected for this analysis were published in recent years, as the focus on implementing AI methods for hydrological and environmental modeling has increased with the recent development of new algorithms. The insights extracted from the analysis of the selected literature can be summarized as follows:

Most analyzed studies used hybrid approaches that combined multiple models to better perform the task at hand. Although success rates varied by model, target parameter, and scenario, most approaches achieved satisfactory, reliable results, indicating the area's maturity. Some AI algorithms also consistently outperformed others for certain water quality indicators. Based on our analysis, the most commonly modeled parameters were total dissolved solids (TDS), Biochemical Oxygen Demand (BOD), and pH, all significant indicators of water quality and likely the most popular "benchmark variables".

Among the studies included in this review, neural-network approaches were the most frequently represented methodological family. Neural networks, in general, are among the most robust at capturing the nonlinear and multivariate relations, but tend to be less interpretable and more computationally expensive. Ensemble tree-based approaches (RF, XGB, etc.) were the second most used AI method, used in 53% of surface water studies and 33% of groundwater studies, offering a balanced approach in terms of accuracy and overfitting resistance, as well as explainability through feature importance.

Metaheuristic and hybrid models appear more effective for noisy time series datasets, and optimization-guided feature selection

was confirmed as a strong performance booster. Fuzzy logic and regression-based AI approaches were less popular than the other techniques. However, they still showed applicability for low-complexity, fast-processing use cases (e.g., TDS and EC modeling in a wetland environment).

Among the reviewed studies, GPR demonstrated particularly strong performance for WQI estimation, with prediction uncertainty below 2% in the Tongi canal study. CEEMDAN-XGB hybrids offered the best noise-filtering capability in pH and turbidity. RF-SVM hybrids performed well in BOD modeling, and EFO-optimized MLPs could offer better DO estimation with faster convergence and lower errors. Fuzzy-neural hybrids (ANFIS) are very useful in TDS and EC estimation in groundwater.

Several open research issues remain, the main one being model interpretability. As most AI models are “black boxes”, the transparency of AI systems is significantly lower, leading to lower stakeholder trust. Another data-side challenge is data sparsity in many areas, which reduces the generality and transferability of developed models to other geographical areas. As a possible step forward in improving on this, this work introduces the XAI-STD (explainable AI-spatiotemporal decomposition) framework that is based on four key components:

- Signal decomposition
- Metaheuristic-based feature selection
- Stacked probabilistic modeling
- Interpretability modules

Such a framework could be more transparent, adaptive, and robust, offering higher-quality water-quality assessments that, with the spread of IoT-based sensor networks, will be more deployable, as the models can be retrained and redeployed in real time.

Future research should move beyond conventional supervised learning toward foundation models and large language models (LLMs), which may facilitate knowledge integration, multimodal data interpretation, and decision-support applications. However, their reliability and domain-specific validation remain important challenges. Integration with Digital Twin technologies could enable continuous coupling between physical water systems, sensor observations, and dynamic AI predictions. Physics-informed AI may improve physical consistency and generalizability by incorporating hydrological and hydrochemical constraints into model development. Federated Learning may provide a pathway for collaborative model development across geographically distributed monitoring networks while reducing the need to centralize sensitive or institutionally restricted data. Together, these approaches may support more transferable, interpretable, physically consistent, and operationally adaptive water-quality prediction systems.

Statements & declarations

Author contributions

Mohammad Khoshnam: Conceptualization, Methodology, Data curation, Visualization, Writing - Original Draft, Writing - Review & Editing.

Hojat Karami: Investigation, Supervision.

Amin Shams: Project administration, Methodology, Validation, Writing - Review & Editing.

Ozgur Kisi: Writing - Review & Editing, Validation.

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