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The influence of quartz powder addition on the production of paving blocks

Keywords: paving blocks, physical, mechanical, classification, quartz powder

Introduction

The construction industry has experienced a notable evolution in recent decades, propelled by the need to satisfy increasing infrastructure demands alongside the necessity of tackling environmental sustainability issues (Ramanna et al., 2024). Contemporary construction methods are progressively focusing on the creation of building materials that fulfill structural performance standards while also aiding in environmental protection and resource efficiency (Shurrab et al., 2019). This significant change has prompted a thorough investigation into alternative materials and cutting-edge construction methods that aim to minimize the environmental impact of infrastructure development, all while preserving or improving both structural and nonstructural integrity and durability (Aslami, 2023).

The historical development of paving materials has been intricately connected to patterns of urbanization and advancements in technology (Arya & Sahu, 2024). Conventional paving techniques, which depended significantly on stone dust and

simple concrete mixtures, have slowly transitioned to more advanced methods that utilize engineered materials and innovative mixing processes (You et al., 2018). The shift from traditional asphalt and concrete pavements to modular paving systems marks an important advancement in construction technology, providing enhanced flexibility, maintenance efficiency, and environmental performance when compared to monolithic paving solutions (Sylvester, 2017).

In modern construction material selection, environmental considerations have taken center stage, especially regarding urban infrastructure development (Abouhelal et al., 2023). The idea of sustainable construction highlights the significance of using materials that reduce environmental impact across their entire lifecycle, from the extraction of raw materials to their disposal or recycling at the end of life (Wang, 2025). This approach has encouraged exploration among experts and practitioners for alternative materials that can lessen dependence on conventional cement-based systems while upholding structural performance standards (Blok et al., 2019). The incorporation of waste materials and by-products into construction applications has surfaced as a promising approach for realizing both environmental and economic advantages (Jaya et al., 2025).

The components of paving blocks or concrete pavers include cement or other binding materials, water, quartz powder, and aggregates. These precast building components maintain their quality despite the presence of additional substances (Megawati, 2018). Currently, paving blocks are widely employed by communities for the development of diverse structures, such as sidewalks, parking lots, and yards. Paving blocks are commonly utilized because they can withstand pressures within specific limits and are easy to install. Furthermore, paving blocks offer distinct advantages over traditional pavements, particularly regarding their environmental benefits as water-permeable surfaces, their potential for mass production, and their visual attractiveness.

Stone dust is generally added to the mix alongside cement, aggregates, and water when producing paving blocks. This additive significantly increases the durability of the block while also enhancing its surface finish. Due to escalating prices, diminishing availability, and heightened demand for stone dust, it is essential to explore alternative options. Quartz powder is classified as an admixture during the mixing process. The pulverized substance is integrated into the paving block mixture to enhance the characteristics of the mix or the paving block (Mulyati & Jovari, 2024).

The strength of paving blocks is notably affected by the inclusion of calcium oxide (CaO) and silica (SiO₂) in the materials utilized for their preparation, including sand. To improve the durability of the paving blocks, fine aggregates that are abundant in silica and calcium, including quartz sand and river sand,

are incorporated (Awolusi et al., 2023). Quartz sand is widely available across all regions of West Kalimantan, with a total accumulation of 630,431,456.75 tons (Dinas Energi dan Sumber Daya Mineral Provinsi Jawa Barat, 2018). Nonetheless, the application of quartz sand as fine aggregate in paving blocks is infrequent, leading to its insufficient utilization. Quartz sand is mainly composed of quartz crystals (SiO_2). Quartz sand consists of various components, including CaO (calcium oxide), SiO_2 (silicon dioxide), Al_2O_3 (aluminum oxide), Fe_2O_3 (iron oxide), TiO_2 (titanium oxide), K_2O (potassium oxide), and MgO (magnesium oxide). When reduced to the powder scale, these oxide materials will display modified properties, which are then designated as powder materials. Upon refinement, quartz sand with a particle size of less than 0.074 mm will undergo alterations in its properties or functions. Throughout the refinement process, the particle size of the sand is diminished from fine aggregate dimensions to a minuscule size nearing the powder scale (Pradhita, 2022). After the quartz powder undergoes fine pulverization, it reveals a significant concentration of SiO_2 . The physical and mechanical properties of paving blocks are expected to improve through the conversion of fine aggregate into powder, recognized as a beneficial additive (Sutandar & Budi, 2024).

Additives are generally divided into two main categories: chemical admixtures and mineral admixtures (Nugroho, 2020). Furthermore, to improve the quality of the paving blocks manufactured in accordance with SNI 03-0691 (BSN, 1996), it is essential to identify the optimal composition of quartz powder to guarantee that the paving blocks produced fulfill the necessary standards (Winanda, 2018). This study will utilize a basic mixture consisting of 1 part cement, 0.2 parts coarse aggregate, and 3.8 parts fine aggregate, derived from earlier studies that yielded Class B quality paving blocks (Milang & Masagala, 2022).

This study aims to develop a formulation for paving block production by substituting stone dust with quartz powder as a mineral admixture (Harimukti, 2022). This formulation will yield Class B paving blocks that adhere to the national standards set forth by Indonesia.

Material and methods

Materials

The materials utilized in this study included cement, fine aggregates, coarse aggregate, and quartz sand.

Cement. Portland composite cement (PCC) has been identified as the primary binder for the production of paving blocks and concrete (Tw et al., 2023). The findings outlined in this study involved the utilization of PCC. Cement is essentially a hydraulic material, as it hardens when combined with water or an aqueous solution, serving as a bonding agent (BSN, 1990). Table 1 outlines that cement consists of three fundamental components with a defined composition.

TABLE 1. Constituents of cement composition

Specification	Quantity [%]
Silicon dioxide (SiO ₂)	23.04
Aluminum oxide (Al ₂ O ₃)	7.40
Ferric oxide (Fe ₂ O ₃)	3.36
Calcium oxide (CaO)	57.38
Magnesium oxide (MgO)	1.91
Sulfur trioxide (SO ₃)	2.0
Loss on ignition (Lo _I)	3.94
Free lime	0.56
Insoluble residue	10.96

Source: own research.

Fine aggregate. The mixture for paving blocks incorporates fine aggregate to improve the strength of the blocks, reduce shrinkage, and lessen the quantity of binders and cement needed (Nofrianto & Hutrio, 2023). Fine aggregate typically consists of natural sand or crushed stone, characterized by a predominant particle size ranging from 0.0625 mm to 2 mm. Fine aggregate is essential in concrete mixes, as it occupies the spaces between coarse aggregates, enhances density, and contributes to the workability and strength of concrete (Nofrianto & Hutrio, 2023). This research indicates that river sand possesses a gritty texture, attributed to its source from the rivers of West Kalimantan. Table 2 presents the physical characteristics of the fine aggregates utilized.

Coarse aggregate. The coarse aggregate is sourced from crushed stone in the Kalimantan Barat region, consisting of stones with a maximum dimension of 0.5 mm. Table 3 presents the physical characteristics of the coarse aggregate used.

Quartz powder. Mineral resources represent finite natural assets that generally encompass multiple valuable minerals (Mulyati, 2023). Alongside the primary minerals, the associated minerals can also possess significant economic potential for extraction. The Mandor River basin hosts numerous alluvial gold deposits

that are being extracted by the local community. Landak Regency possesses a remarkable abundance of potential mineral resources. Ilmenite (TiO₂), magnetite (Fe₂TiO₄), and zircon (ZrSiO₄), along with various other minerals, are commonly found in alluvial gold deposits. The utilization of these accompanying minerals, which encompass rare earth elements (REE), has the potential to enhance product quality across various sectors, including metal, information technology, electronics, oil and gas, and nuclear energy. As a result, there is an anticipated rise in global market demand for REE.

TABLE 2. Characteristics of the fine aggregate

Specification	Quantity
Specific gravity [kg·m ⁻³]	2.646
Fine modulus [-]	3.030
Bulk density [kg·m ⁻³]	1,523.65
Fine particles less than 0.0075 mm [%]	5.43

Source: own research.

TABLE 3. Characteristics of the coarse aggregate’s physical properties

Specification	Quantity
Absorption [%]	0.188
Bulk density [kg·m ⁻³]	1.668
Water content [%]	0.272
Fine modulus	4.9
Aggregate abrasion [%]	12.21
Mud content [%]	2.4
Specific gravity [kg·m ⁻³]	2.455

Source: own research.

Quartz is found in the research domain as grit that is dispersed within the alluvial deposits, currently associated with artisanal gold mining residues. The sediments in the study area consist of a combination of clay and silica sand (Falah & Muzaki). Quartz powder presents as a whitish-gray substance characterized by granules that range from fine to coarse in texture. The composition includes whitish-green elements of igneous rock and silt (Nabila et al., 2023). The resources of quartz powder are calculated by taking the depth of each borehole, multiplying

it by the area of influence associated with each borehole, and subsequently multiplying that result by the specific gravity of quartz. The results of the expected quartz resource assessments at each study site are as follows: The total quantity of quartz sand resources across the five regions of Mandor is 5,245,400 tons, covering an influence area of 322,290 m² (32.23 ha). The quartz sand in the study area is notably plentiful and typically devoid of impurities such as soil and silt, having undergone a purification process during the gold mining operations. Currently, the local community employs quartz powder as a fundamental resource for construction purposes (Umurrudin et al., 2018). Table 4 presents the physical properties of quartz powder. The physical properties of quartz sand converted into quartz powder are detailed in Table 5.

TABLE 4. Results of the chemical analysis of quartz powder

Specification	Quartz powder content [%]	
	samples from Mandor riverbank	samples from Mandor riverbank
Silicon dioxide (SiO ₂)	98.6	98.8
Aluminum oxide (Al ₂ O ₃)	0.51	0.29
Ferric oxide (Fe ₂ O ₃)	0.25	0.25
Ilmenite dioxide (TiO ₂)	0.18	0.15
Calcium oxide (CaO)	0.21	0.21
Potassium dioxide (K ₂ O)	0.012	0.028
Magnesium oxide (MgO)	–	–
Free calcium oxide	–	–
Water (H ₂ O)	–	–

Source: own research.

TABLE 5. Physical properties of quartz sand

Specification	Quantity
Bulk density [kg·m ⁻³]	1.760
Organic content [%]	3
Water content [%]	4.55
Mud content [%]	2.88

Source: own research.

The water utilized in this study is sourced from local water providers (PDAM). The pH of municipal water ranges from 6 to 7, indicating that it is in excellent condition and complies with the quality standards for potable water. This water serves as a crucial component in the preparation of paving block materials, including cement, fine aggregate, coarse aggregate, and quartz powder. The quality of water is crucial for maintaining the integrity of the mix and the strength of the final product, adhering to the established standards for paving block production.

Mix proportions

Figure 1 demonstrates the methodology for material combination, whereas Table 6 details the mix proportions employed in the experimental procedures. This study utilizes quartz powder as an alternative to stone dust.

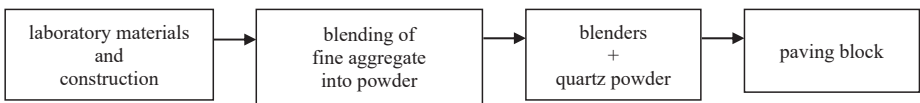


FIGURE 1. Stepwise procedure for the paving block production
Source: own research.

TABLE 6. Mix design for paving blocks (volume of 1 m³)

Variation	Cement	Fine aggregate	Coarse aggregate	Quartz powder	Water to cement ratio (w/c)
0%	1	3.8	0.2	0	0.3
10%	1	3.8	0.2	0.1	0.3
20%	1	3.8	0.2	0.2	0.3

Source: own research.

Laboratory tests

Visual test. Following the casting of the paving blocks, a curing duration of 28 days is necessary. During the testing phase, it is essential that the paving blocks are in a dry state. The measurements of the paving blocks are taken in terms of length, width, and thickness. The examination of the paving block’s surface encompasses the state of the corners, density, and overall surface condition.

Bulk density test. The paving block undergoes a drying process in an oven for a duration of 24 hours to achieve a dehydrated state. Once the required data has been gathered, the calculation of the sample's bulk density is performed using the subsequent equation:

$$\text{bulk density} = \frac{W}{V},$$

where: W – weight of paving block, V – volume of paving block.

Water absorption test. A water absorption test was conducted following BS 1881-122 (BSN, 2011) to determine the amount of water absorbed under the specified conditions. Following a period of 28 days, the paving block samples underwent an oven-drying process at 1,100°C for a duration of 24 hours. Following this, the paving block was completely immersed in water for a duration of 24 hours after establishing the oven-dry weight. The measurement of weight after water immersion was also performed.

Wear test. The wear testing process is carried out to assess the capacity of the paving block to endure the friction present on the surface layer across each variation of the paving block. The testing is conducted once the paving block has matured for 28 days.

Compressive strength test. The compressive strength of the samples was evaluated at 7 days, 14 days, 21 days, and 28 days following ASTM C39 (ASTM, 2023) and ALCONPAT protocols, utilizing the universal testing machine. The paving block samples were laid flat on their full face, and the compressive strength value was determined by dividing the maximum test load (up to the point of rupture) by the cross-section of the paving block specimen.

Results and discussion

Determination of the bulk density of paving blocks

Once the cast paving block samples are fully dried, the testing phase can begin. Prior to testing, the paving blocks undergo a conditioning process involving water application, which guarantees that the sun-dried blocks maintain a uniform moisture content. Following this, the paving blocks were assessed for weight in a dry state after a curing period of 28 days.

Figure 2 depicts the relationship between age and bulk density across three different conditions: 0% quartz powder (blue), 10% quartz powder (orange), and 20% quartz powder (gray). At 3 days of age, the sample containing 10% quartz powder (orange) demonstrates the highest bulk density, followed by the 20% quartz powder (gray), whereas the 0% quartz powder (blue) displays the lowest bulk density. At 7 days of age, the bulk density was still elevated at 20% quartz powder, while the 10% variant showed a slight increase compared to the 0% variant. From 14 to 28 days, the variations begin to converge, leading to a stabilization in bulk density; nonetheless, the 20% quartz powder consistently shows the highest values across various intervals. In the initial stages of interpretation (3 days and 7 days), the samples containing 10% and 20% quartz powder demonstrated higher bulk densities compared to those with 0% quartz powder.

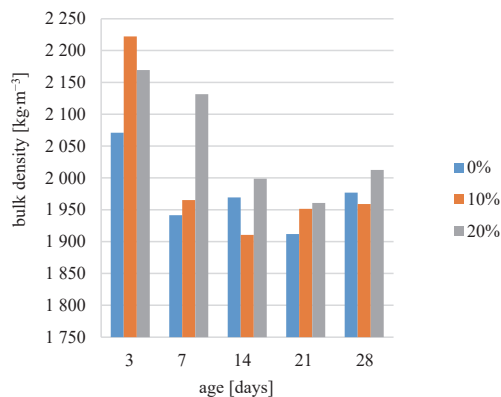


FIGURE 2. Relationship between bulk density and age for all variations of paving blocks

Source: own research.

This indicates that, during the early stages, the incorporation of extra quartz powder has a beneficial effect on bulk density. As the age advances from 14 days to 28 days, the disparities among the variations lessen, indicating that the initial influence of quartz powder begins to wane or stabilize over time. Samples containing 20% quartz powder demonstrated an increased bulk density at various ages, indicating that in these circumstances, the presence of additional quartz powder had a more pronounced effect compared to other conditions. The variations of 10% and 20% lead to an increase in bulk density during the early stages, especially noticeable at 3 days and 7 days. In the later stages (14–28 days), the variations show

minimal differences; however, the 20% quartz powder continues to demonstrate higher values at certain points. For maintaining a higher bulk density in the long term, incorporating 20% quartz could prove to be more beneficial.

Compressive strength determination

The compressive strength results of the quartz particle variations for paving blocks are illustrated in Figure 3. The compressive strength increases with the curing days, as anticipated, in the range of 13.03–18.69 MPa, 15.63–23.60 MPa, and 16.23–23.61 MPa for the 3-, 7-, 14-, 21-, and 28-day curing periods, respectively. The paving blocks containing different quantities of quartz powder exhibited the greatest compressive strength compared to the other mixtures tested; nonetheless, utilizing quartz powder as a binder did not lead to a noteworthy enhancement in compressive strength. The reduction in strength observed in certain variations can be attributed to the development of less robust interfacial transition zones (ITZ) between the quartz powder and the aggregate. Nonetheless, paving blocks that incorporate quartz powder demonstrate superior compressive strength when compared to those that lack this modification. The maximum compressive strength observed for paving blocks containing 20% quartz powder was 23.61 MPa. Paving blocks that achieve a compressive strength of 20 MPa or greater at 28 days are classified as Class B under Indonesian standards. These blocks are suitable for applications in parking lots, pedestrian zones, and other areas with light loads.

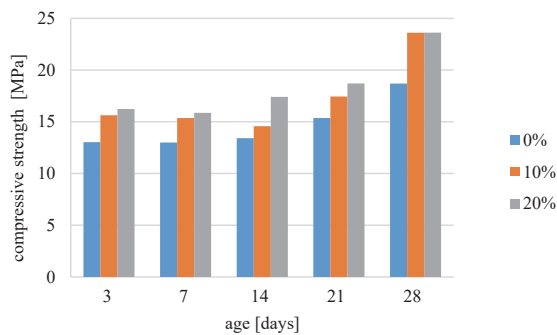


FIGURE 3. Relationship between compressive strength and age for all variations of paving blocks
Source: own research.

The analysis of variance (ANOVA) performed to assess the effect of quartz powder addition on the compressive strength of paving blocks yields strong statistical evidence concerning the experimental results. The computed F-statistic of 1.574 is significantly lower than the critical F-value of 3.89 at the established significance level of $\alpha = 0.05$, suggesting that the differences observed among the treatment groups (0%, 10%, and 20% quartz powder addition) lack statistical significance. This finding is supported by the p-value of 0.247, which is above the established alpha threshold, resulting in the retention of the null hypothesis (H_0) that posits no significant difference between the mean compressive strengths of the various mixture compositions. The inter-group mean square value of 14.90, when compared to the intra-group mean square of 9.47, indicates that the variation within groups is relatively comparable to the variation between groups. This reinforces the conclusion that the addition of quartz powder at the tested concentrations does not significantly affect the mechanical properties of the paving blocks.

From a practical engineering perspective, this statistical outcome suggests that the use of quartz powder as a partial substitute for traditional aggregates maintains the structural integrity of the paving blocks, thereby endorsing its potential as a sustainable alternative material. The economic implications of these findings are notably important, as the use of quartz powder – a waste material with limited commercial value – at a 10% replacement level could greatly lower manufacturing costs while maintaining the product’s mechanical performance, thereby offering an environmentally and economically sound solution for paving block production.

The data indicates that all variants fall under the Class B category of paving blocks as per the quality standards outlined in SNI 03-0691 (Table 7).

TABLE 7. Compressive strength of paving blocks

Variation	Compressive strength at 28 days of age [MPa]	SNI 03-0691 (paving block classification)				Class
		A	B	C	D	
0%	18.69	35	17	12.5	8.5	B
10%	23.60	35	17	12.5	8.5	B
20%	23.61	35	17	12.5	8.5	B

Source: own research.

Figure 4 demonstrates the relationship between compressive strength and the percentage of quartz powder. The graph clearly illustrates that the compressive strength rises with an increase in quartz powder percentage until it hits a peak, after which there is a slight decline.

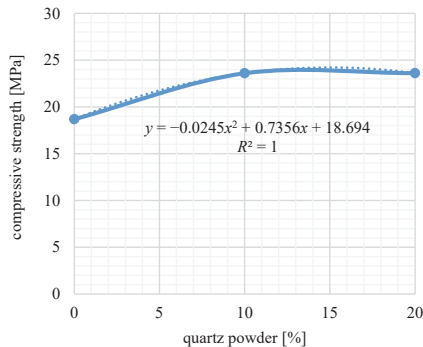


FIGURE 4. Relationship between compressive strength and quartz powder percentage for all variations of paving blocks

Source: own research.

The model utilized demonstrates a strong alignment with the existing data, as indicated by the quadratic regression equation: $y = -0.0245x^2 + 0.7356x + 18.694$, with an R^2 value of 1. This equation effectively accounts for all data points in a satisfactory manner. The addition of quartz powder leads to an increase in compressive strength, as demonstrated by the pattern observed in the graph, ultimately peaking at around 10%. The compressive strength value subsequently shows a slight reduction as the quartz powder content rises to 20%. At a quartz powder level of around 10%, the peak compressive strength reaches approximately 25 MPa. As a result, the optimal level for compressive strength is determined to be 10% quartz powder. It is essential to assess whether the extent of the reduction at elevated levels is significant or if it stays within an acceptable limit beyond the optimal point, given that an increase in quartz powder leads to a slight decline in compressive strength.

Absorption determination

The water absorption capacity for each variation of paving block is assessed via absorption testing, conducted once the paving block has matured for 28 days.

The absorption values presented in Table 8 indicate the average absorption percentage of the paving block variations that utilize quartz powder in place

of stone dust. The 20% variation demonstrated the lowest absorption value at 6.78%. The 0% variation exhibited the highest absorption value of 9.96%, as established through the tests and calculations performed on all variations of the paving blocks. The absorption test performed according to SNI 03-0691 resulted in Quality C for paving blocks containing 10% and 20% quartz powder, while those with 0% quartz powder received a Quality D rating.

TABLE 8. Absorption of paving blocks

Variation	Present absorption [%]	SNI 03-069 (paving block classification)				Class
		A	B	C	D	
0%	9.96	3	6	8	10	D
10%	7.84	3	6	8	10	C
20%	6.78	3	6	8	10	C

Source: own research.

Figure 5 demonstrates the relationship between the percentage of quartz powder and absorption levels. The observed pattern indicates a decrease in the absorption value with an increase in quartz powder. This suggests that the capacity of the paving blocks to absorb water is diminished due to variations or inconsistencies in the quartz powder. The equation $y = 0.0053x^2 - 0.265x + 9.96$, derived from the graph, indicates that this relationship is quadratic or curved instead of linear. The decrease in absorption is not uniform; rather, it exhibits a distinct pattern that decelerates as the proportion of quartz powder increases.

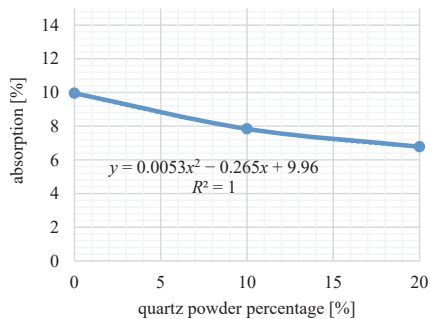


FIGURE 5. Relationship between absorption and quartz powder percentage for all variations of paving blocks

Source: own research.

The R^2 value of 1 clearly indicates that this equation precisely fits the data. This demonstrates that every tested data point aligns perfectly with the equation, exhibiting no discrepancies. In summary, the water absorption capacity of paving blocks could be diminished by an increased percentage of quartz powder or other influencing factors, such as the method employed. This can serve as an essential consideration in the development of materials that require minimal water absorption, like paving blocks. This graph clearly illustrates the significant role that quartz powder percentages play in maintaining optimal absorption values.

Wear determination

When the paving blocks reach 28 days of age, wear testing is conducted to ascertain their capacity to tolerate friction on the surface layer in each variation of paving blocks (Table 9).

TABLE 9. Wear of paving blocks

Variation	Wear [mm·min ⁻¹]	SNI 03-0691 (classification)				Class
		A	B	C	D	
0%	0.046	0.103	0.149	0.184	0.251	A
10%	0.021	0.103	0.149	0.184	0.251	A
20%	0.004	0.103	0.149	0.184	0.251	A

Source: own research.

The wear value in Table 9 represents the average wear percentage of the paving block variations with quartz powder as a substitute for stone dust. In the wear test, all variations of the paving blocks are classified as Quality A according to SNI 03-0691.

Figure 6 illustrates the correlation between age and wear. The observed pattern indicates that wear diminishes with an increase in the level of quartz powder. This indicates a particular effect that reduces the degradation of the paving block material. The graph has produced the equation $y = 5E-05x^2 - 0.003x + 0.0461$, demonstrating a quadratic relationship. This suggests that the effect of quartz powder on wear is characterized by a curvilinear relationship rather than a linear one. With an increase in the level of quartz powder, the impact of reduced wear becomes increasingly pronounced. Furthermore, the equation’s ability to precisely depict the existing data is evidenced by an R^2 value of 1. In summary, all the tested data points conform to the established pattern of the equation, without any exceptions. The findings indicate that increasing the percentage of quartz powder in the paving

blocks could potentially reduce wear, highlighting a practical approach to enhancing durability. This graph illustrates a strong relationship between wear and quartz powder concentrations, indicating that paving blocks show diminished wear levels as quartz powder increases. This could serve as a basis for designing or optimizing the production of paving blocks to enhance durability and minimize erosion.

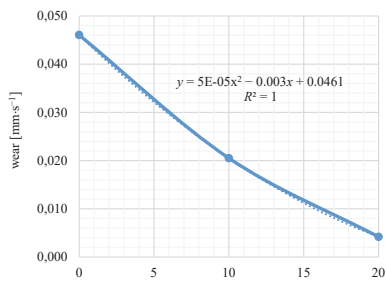


FIGURE 6. Relationship between wear and quartz powder percentage for all variations of paving blocks
Source: own research.

Visual testing of paving blocks

Paving blocks are subjected to a thorough visual inspection to ensure they are level and devoid of any defects or cracks and that their corners and edges can withstand manual handling without breaking. The thickness tolerance of the paving block’s dimensions is approximately 8%. The test is performed once the paving block has attained an age of 28 days (Table 10).

TABLE 10. Visual condition of paving blocks

Variation	Description	Sample condition	According to SNI 03-1691-1996
0% 10% 20%	STRUCTURE		
	flatness	flat	flat
	crack	no crack	no crack
	surface texture	rough	smooth
	FLANK		
	geometry	geometry	geometry
	sharpness	sharp	sharp
	strength	strong	strong

Source: own research.

The findings from the experiments carried out (Table 11) indicate that all variations meet the visual and physical criteria specified in SNI 03-0691.

TABLE 11. Size of paving blocks

Variation	Thickness condition [mm]	Thickness [mm]	Tolerance	Length [mm]	Width [mm]	According to SNI 03-1691-1996
0%	64.20	60	±8%	210	105	fulfilled
10%	59.50	60		210	105	fulfilled
20%	60.78	60		210	105	fulfilled

Source: own research.

Conclusions

The thorough experimental study of incorporating quartz powder in the production of paving blocks reveals considerable potential for real-world applications within the construction sector. The findings clearly demonstrate that the ideal 10% quartz powder variation provides exceptional performance traits that directly contribute to improved field applicability and economic feasibility. The decreased bulk density of paving blocks with 10% quartz powder offers significant benefits regarding transportation efficiency, ease of installation, and lower structural loading requirements. This makes them especially appropriate for large-scale infrastructure projects, where material handling costs represent a considerable share of total project expenses. The attainment of Class B compressive strength in all variations guarantees structural integrity for standard paving uses, such as pedestrian pathways, residential driveways, and light commercial surfaces, thus fulfilling essential engineering standards for practical application.

The significant enhancement in water absorption properties, with the 10% and 20% quartz powder variations reaching Class C absorption as opposed to Class D for traditional mixtures, suggests improved durability and longevity under field conditions, especially in tropical climates where moisture-related degradation presents considerable challenges to pavement performance. The outstanding resistance to deterioration and attrition exhibited by all variations, along with satisfactory visual quality traits such as flat surfaces, a crack-free appearance, and accurate geometric dimensions, guarantees that these paving blocks will preserve their aesthetic and functional properties in typical field service conditions.

The dimensional stability and thickness compliance across all variations further confirm the manufacturing consistency necessary for effective field implementation, tackling quality control issues that are essential for large-scale production and deployment.

From a practical implementation standpoint, identifying 10% quartz powder as the optimal variation presents a strong argument for immediate field application. This approach offers the advantages of reduced material weight, enhanced moisture resistance, preserved structural integrity, and potential cost savings through the utilization of waste materials. The results offer construction professionals empirically supported information to aid in the specification and implementation of quartz powder-enhanced paving blocks in practical applications, enhancing both sustainable building practices and pavement performance benchmarks. The study lays the groundwork for increasing production and putting in place quality control measures essential for the effective commercial use of this groundbreaking paving block technology.

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Summary

The influence of quartz powder addition on the production of paving blocks.

The research investigates how quartz powder addition affects the physical and mechanical properties of concrete paving blocks made from cement, fine aggregate, water, and coarse aggregate mixtures. Quartz powder was evaluated as an as an effective waste material alternative to conventional stone dust, with testing conducted on compressive strength, abrasion resistance, bulk density, absorption, and visual inspection parameters. The findings demonstrate that the optimal 10% quartz powder addition significantly improves paving block properties, achieving Class B according to Indonesian National Standard SNI 03-069, which requires a minimum compressive strength of 17 MPa and designates suitability for pedestrian traffic areas, residential walkways, and light vehicular applications. The enhanced performance is attributed to the high silica content in quartz powder, which strengthens cementitious matrix bonds, while Class B ensures compliance with construction specifications for moderate-stress environments, providing adequate durability and structural performance for typical urban infrastructure applications. This research validates quartz powder as a viable, waste material admixture that maintains required quality standards while potentially reducing manufacturing costs through waste material utilization.

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A new approach for assessing landslide vulnerability at the urban scale: the case of the city of Constantine, Algeria

Keywords: landslide, vulnerability, risk assessment, Constantine, multi-criteria analysis

Introduction

Defined in various ways, risk is an assessment of the potential for harm based on the probability of occurrence, level of intensity, and other parameters that characterize an event and its consequences. Risk can thus be understood as the combination of an impending event's likelihood to happen (hazard) and facing its consequences (problem & vulnerability) (Wisner et al., 2004).

Vulnerability is defined as the conditions determined by physical, economic, social, and environmental factors or processes that increase the susceptibility of a community to the effects of hazards (Birkmann, 2006).

Landslides represent a significant natural hazard in many urban regions, particularly where complex geological, climatic, and anthropogenic factors intersect. In Algeria, Constantine is one of the cities most exposed to this risk due to its fragile geological structure, rapid urbanization, and variable climate. Although some regional studies exist regarding landslide susceptibility, there

has yet to be an integrated multi-factor evaluation of urban-scale analysis for comprehensive regional vulnerability assessment. Given the strategic significance landslides hold from both humanitarian and financial perspectives, bolstering risk prevention while simultaneously decreasing vulnerabilities at both a regional and local level becomes essential. This proactive approach aids in estimating danger while assessing vulnerability (Glade, 2003; Fell et al., 2008).

This study proposes a new approach for assessing landslide vulnerability in Constantine using a multi-criteria evaluation method integrated into a GIS. Six key parameters – topography, geology, hydrology, rainfall, seismicity, and land use – were weighted based on expert surveys and mapped to generate a composite vulnerability index (Alcántara-Ayala, 2002).

The resulting vulnerability map categorizes Constantine into three vulnerability levels (high, medium, low). The areas with the highest vulnerability correspond closely with known zones of historical instability. A comparative validation with the reference ARCADIS-Simecsol study shows a 97% spatial match, confirming the reliability of the proposed model.

The study of the various indices of landslide vulnerability, including geology, hydrology, slope, and climatology, allows us to determine the influence of each factor in the study of instability in the city of Constantine (Bourenane et al., 2015; Bourenane et al., 2016; Abdi et al., 2021).

Our contribution is to use a new multi-criteria approach to produce a detailed map of landslide vulnerability in the city of Constantine using a GIS, as understanding the contribution of each factor is essential to assess the phenomenon accurately (van Westen et al., 2008). By identifying spatial patterns of vulnerability (Papathoma-Köhle et al., 2017), this approach provides valuable guidance for decision-makers in prioritizing interventions and planning future development.

Material and methods

The first stage of any research is to collect information about the study area in a structured way. This begins with an analysis of the natural processes associated with the physical environment that are responsible for the risk, followed by a characterization of the lithology and identification of signs of instability on the slopes (Guzzetti et al., 2005; van Westen et al., 2006).

Geographical location

Constantine is Algeria's third largest city, located in the east of the country between 36.24° latitude and 6.60° longitude, about 437 km from Algiers (Fig. 1). It covers an area of 2,297.2 km² and in the last census in 2008, it had a population of 921,893, which is now estimated at around 1.5 million (Office National de la Météorologie [ONM], 2024).

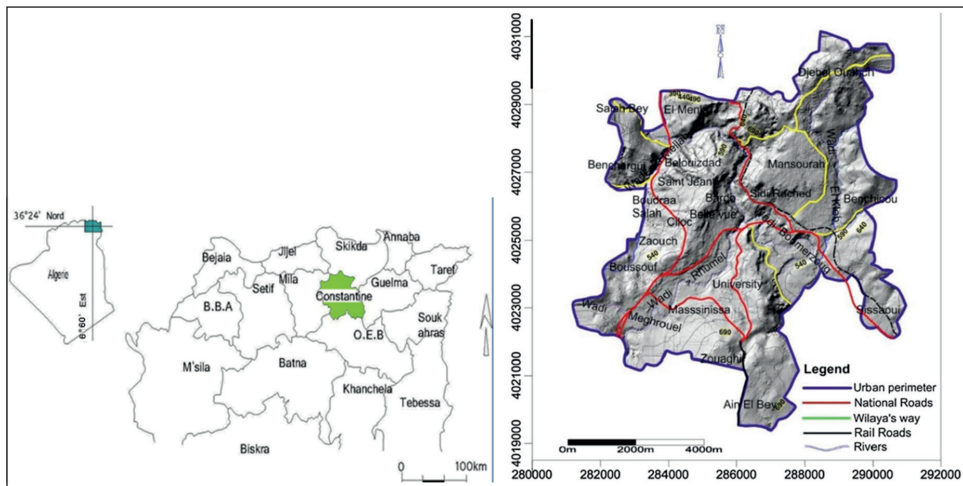


FIGURE 1. Geographical location of the city of Constantine

Source: own work based on Bourenane and Bouhadad (2021).

Causative factors of landslides

Causative factors – also referred to as conditioning or predictive variables – play a fundamental role in the initiation and propagation of landslides. These factors can be categorized into six main groups, each representing different dimensions of the natural and anthropogenic environment that influence slope stability (Guzzetti et al., 2005, 2007; van Westen et al., 2008; Arif et al., 2023).

1. Geomorphological factors

These include terrain elevation (hypsoetry) and slope angle, which directly control the gravitational forces acting on a slope. Steep slopes increase the downslope component of gravity, thus enhancing landslide susceptibility (Yalcin, 2008). Slope curvature and aspect also influence soil moisture retention and solar radiation exposure (Taib et al., 2025).

2. Hydrological factor (drainage density)

High drainage density areas often experience rapid water flow, leading to slope undercutting and erosion. The proximity to drainage networks is frequently used in GIS models as a landslide trigger proxy (Saha et al., 2002; Lee & Pradhan, 2006).

3. Climatic factor (precipitation)

Rainfall is one of the most important external triggers for landslides, particularly in regions with intense or prolonged precipitation. Rainfall intensity and antecedent moisture conditions affect pore water pressure and slope saturation (Glade, 2003; Guzzetti et al., 2007).

4. Geological factors (lithological facies and fracturing)

Weak or weathered lithologies such as clay, marl, or shale are more susceptible to failure than consolidated rock types. Additionally, the degree of fracturing or faulting influences groundwater movement and slope weakening (Mezhoud & Benazzouz, 2018).

5. Seismic factor (seismic micro-zonation)

Ground shaking from seismic events can directly trigger landslides, especially in areas already predisposed by geological or geomorphological conditions. Micro-zonation maps are used to identify zones of amplified seismic risk (Chettah et al., 2024).

6. Anthropogenic factors (land use and human activity)

Deforestation, road cuts, urbanization, and slope modification for agriculture can dramatically alter natural slope stability. Human-induced changes are increasingly considered in landslide susceptibility studies, especially in expanding urban regions (Pradhan, 2010).

The topography of Constantine. The city of Constantine is in the high eastern plains of the Tellian Atlas, at an average altitude of between 500 m and 800 m. Its topography is undulating, with flat areas bordered by steep slopes and marked differences in level. Its varied and contrasting topography includes deep gorges, plateaux, valleys, and hills surrounding the rocky outcrop, which forms the central core of the town (Mezhoud & Benazzouz, 2018).

According to the slope map (Fig. 2), the distribution of slopes shows that:

- More than half of the area studied has slopes between 10 and 30%.
- The steepest slopes, over 30%, are mainly found in the vicinity of rivers, forming their banks.
- The lowest slopes (0–10%), which account for a large part of the PDAU, are found on the alluvial terraces of the Rhumel and Boumerzoug wadis and, to the north-west, on the rocky outcrop, and the Mansourah, Coudiat, and Salah Bey plateaux.

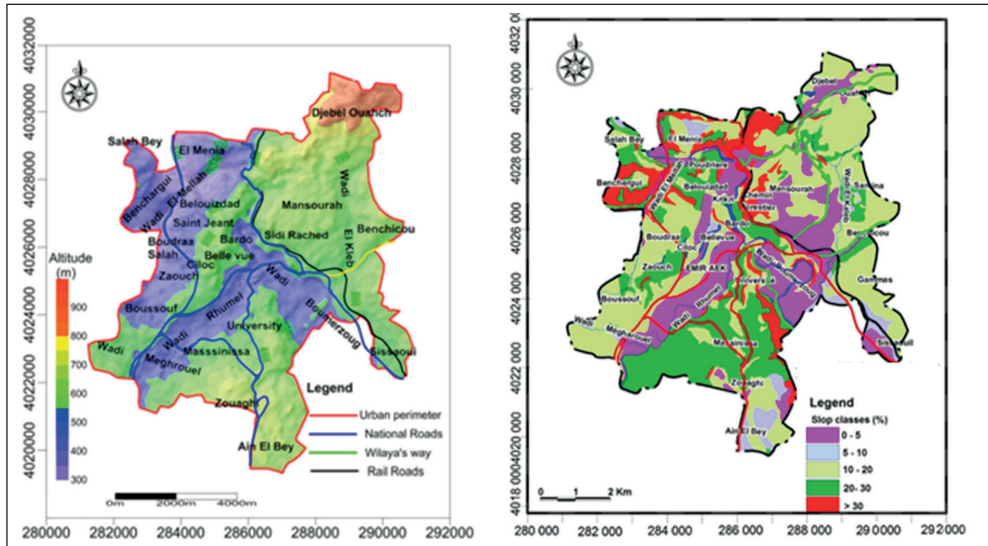


FIGURE 2. Altimetry and slope maps of the city of Constantine

Source: Bourenane et al. (2015).

The hydrology of Constantine. The region of Constantine is characterized by a relatively dense hydrographic network (Fig. 3), with the city itself situated at the confluence of two main watercourses:

1. The Oued Boumerzoug, which flows north-south upstream and then north-west-south-east downstream, and the Oued Rhumel, whose direction varies throughout its course, crossing the Constantine gorges.
2. Intense erosion in the region has created numerous tributaries to these two ouadis. These include the Oued Athmania, Oued Seguen, Oued Ziad, and Oued Smendou, which flow into the Oued Rhumel, while the Oued El Klab, Oued Melah, Oued Berda, and Oued Hamimime flow into the Oued Boumerzoug.

It is important to note that these watercourses generally have low flow rates and can even dry up completely during periods of intense heat. On the other hand, during the rainy season, intense rainfall often leads to flash floods that reach record levels and cause landslides along their banks.

Precipitation. In the study, a recent rainfall series covering the period from 1 January 1975 to 31 December 2017, provided by the National Meteorological Office (ONM) in Aï n El Bey, was used for precipitation determination. The city of Constantine is in an area where annual rainfall varies between 400 mm and 800 mm (Fig. 4).

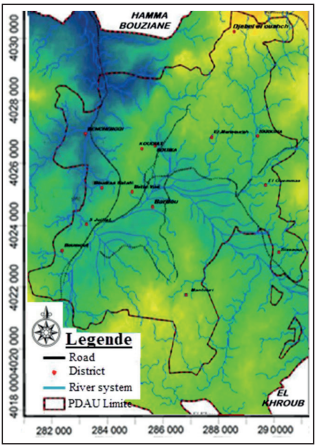


FIGURE 3. Constantine hydrographic network
Source: own work based on Mezhoud and Benazzouz (2018), Samy et al. (2019).

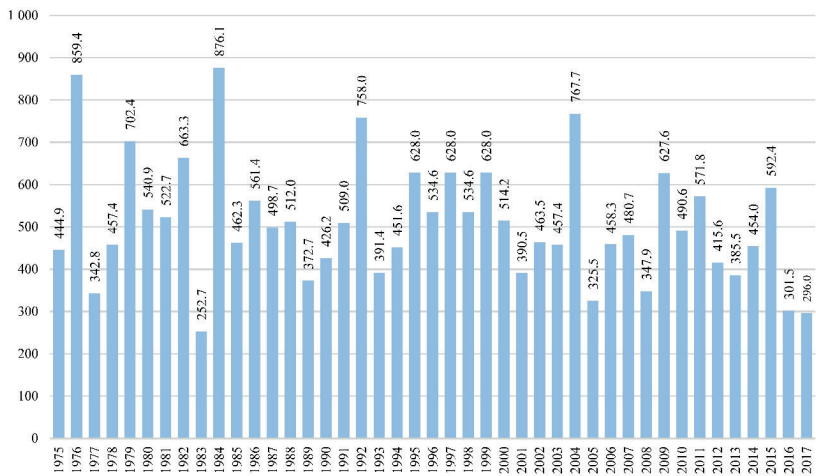


FIGURE 4. Precipitation in Constantine in 1975–2017
Source: ONM (2024).

Local geology. The geological framework of Constantine plays a critical role in assessing landslide risk, with lithological and structural characteristics serving as primary indicators. Based on the work of Coiffait (2012), the study area lacks Pliocene formations; instead, the lacustrine limestones of Ain El Bey and Mansourah have been reclassified as early Quaternary. These are distinguished by a sandy-conglomeratic base and thinner profiles compared to true Pliocene deposits.

The Quaternary formations are primarily composed of lacustrine limestones from the Mansourah, Ain El Bey, and Salah Bey plateaus, along with alluvial terraces from the Rhumel and Boumerzoug rivers (Fig. 5). These formations exhibit varied lithological sequences, including conglomerates, micritic and travertine-like limestones, and cemented sands, each contributing to slope stability characteristics.

Three distinct alluvial terraces are identified:

1. The younger terrace, adjacent to rivers, is flood-prone due to its sandy-silt composition.
2. The intermediate terrace, used increasingly for development, shows moderate flood exposure.
3. The ancient terrace, elevated and composed of conglomerates, is less exposed but reflects tectonic uplift.

The stepped configuration of these terraces suggests significant Neogene tectonic activity, where differential uplift – not river incision – has shaped the landscape and influenced current geomorphological instability (Bouragba et al., 2023).

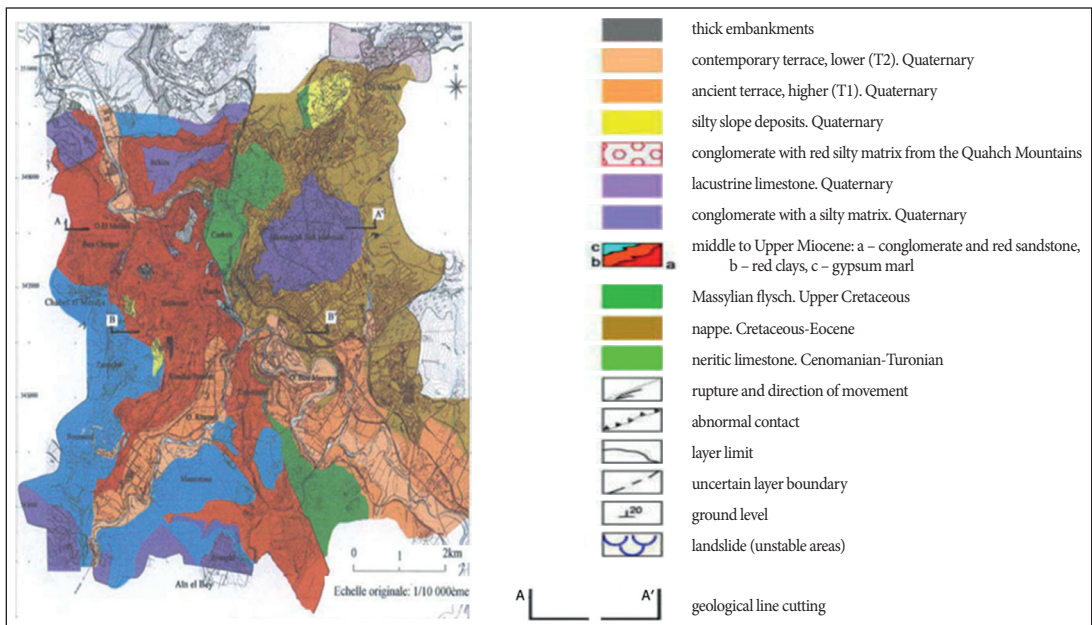


FIGURE 5. Constantine geomorphology

Source: Geological survey; Bougdal-Belhai; USTHB-FST-GAT-DUC Constantine (2002).

Seismicity of the city of Constantine. The seismic history of the Constantine region is essential for understanding local seismic hazards. Historical seismic data help identify potential earthquake sources and active fault zones. Key reference catalogs used in the analysis include works by Harbi (2001, 2003, 2007), Benouar (1994), Mokrane et al. (1994), and international databases such as those from the International Commission on Stratigraphy, as well as studies by Bezzeghoud et al. (1996).

The historical seismicity map was developed through a rigorous methodology involving the compilation and homogenization of various seismic catalogs using standardized magnitude and intensity scales. Major earthquakes, such as those of 1908, 1947, and 1985, serve as benchmarks in the regional seismic record.

Spatial analysis of epicenters reveals concentrated seismic activity along the Constantine–Guelma axis in the Tellian region, the Tébessa area of the Atlas Mountains, and the coastal corridor between Jijel and Skikda – highlighting these as zones of heightened seismic risk in northeastern Algeria.

Demography and spatial distribution of buildings in Constantine. Constantine has undergone significant demographic changes marked by rapid population growth and shifting urban dynamics. Between 1954 and 1966, the city experienced a peak annual growth rate of 9.6%, largely driven by rural migration and natural population increase following Algeria’s independence. Subsequent decades saw continued growth: from 240,000 inhabitants in 1966 to over 450,000 by 1987, fueled by industrial and service sector expansion.

However, from 1987 to 1998, the growth rate slowed to 0.6%, with urban sprawl shifting toward surrounding municipalities due to land shortages within the city. Between 1998 and 2008, Constantine saw a slight population decline, primarily due to outmigration from the urban core, degradation of old housing, and land scarcity.

Since 2008, the population trend has reversed, with growth reaching an estimated 475,510 by 2024, attributed to peripheral urbanization and new housing developments. Over the 1998–2024 period, the population increased by 7.2%.

Housing density remains uneven across the city’s districts, with high density in areas like Sidi Rached and significantly lower values in Kitouni, El Gammas, Les Mûriers, and the 5 July 1962 sector, reflecting variations in housing typology and urban planning.

Survey methodology

To identify the most critical factors influencing the vulnerability of the city of Constantine to landslides, a structured survey was conducted among members of the scientific community, including researchers, academics, and experts in geosciences and natural hazards. The objective of the survey was to prioritize six key parameters: topography, hydrology, rainfall, geology, seismicity, and land use (Ladjel et al., 2025).

The data collected from the survey were systematically analyzed to determine the relative importance given to each parameter by the respondents. The analysis provides valuable insights into experts' perceptions of the key factors contributing to landslide vulnerability in Constantine. The results not only highlight the most influential parameters but also provide a basis for the development of targeted risk reduction strategies. The breakdown of the results is shown in Figure 6.

The results indicate that rainfall is perceived as the most significant factor in triggering landslides, accounting for 26% of responses. This outcome aligns with the understanding that precipitation, particularly when intense or prolonged, plays a critical triggering role by saturating the soil with water, thereby reducing its cohesion and increasing its instability.

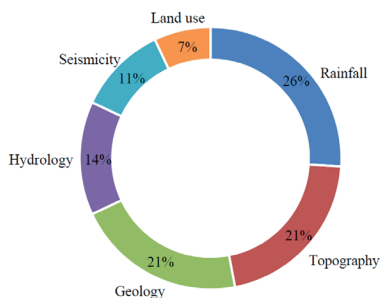


FIGURE 6. Hierarchy of the different factors influencing landslide vulnerability

Source: own work.

The results indicate that rainfall is perceived as the most significant factor in triggering landslides, accounting for 26% of responses. This outcome aligns with the understanding that precipitation, particularly when intense or prolonged, plays a critical triggering role by saturating the soil with water, thereby reducing its cohesion and increasing its instability.

Topography and geology follow closely, each contributing 21%. Topography, through the slope of the land, directly impacts the severity of mass movements, while the geological composition of soils and rock formations dictates their mechanical strength. As such, these two parameters are structural in nature and determine the terrain's susceptibility to movement. Hydrology, with 14%, occupies an intermediate position. It is concerned with the presence and behavior of both surface and groundwater, which can weaken the soil at various depths. While its influence is often indirect, it plays a pivotal role, especially in areas with high infiltration rates. Seismicity (11%) and land use (7%) are considered less influential by the respondents. While seismic activity can indeed trigger landslides under specific geological conditions, its effect is typically localized, linked to particular events. Land use, although seen as a secondary factor, can exacerbate vulnerability through soil overload or changes in vegetation cover.

Development of the approach followed

In this approach, we will produce a predictive map of landslide risk in the city of Constantine, highlighting the main thoroughfares and the most vulnerable areas. This study will be refined by using the vulnerability index method.

The methodology consists of separately mapping the natural and anthropogenic factors that serve as indicators of vulnerability. For each map, we have assigned a critical index on a numerical scale based on its importance, as determined in advance by our survey. By summing these indices, we obtain the map of the total index, which provides a minimum and maximum range of risks that are likely to cause harm or damage to the area studied.

The slope factor. The topography of the site is represented by the slope index (SLI). This index allows the creation of a slope factor map (Fig. 7), which is made using the topographic map of the city of Constantine. The slope classes of the terrain slope parameter are shown in Table 1.

The hydrology factor. The hydrology of the site is represented by the hydrology index (HI). This index allows the creation of a hydrology factor map, which is made using the topographic map and the map of watercourses of the city of Constantine. This index is based on the proximity or distance of the wadis and watercourses, as areas near watercourses are more vulnerable to landslides and erosion effects. The classes of this “terrain hydrology” index are shown in Table 2. Distance from the river indicates the influence of the river on slope stability (Fig. 8).

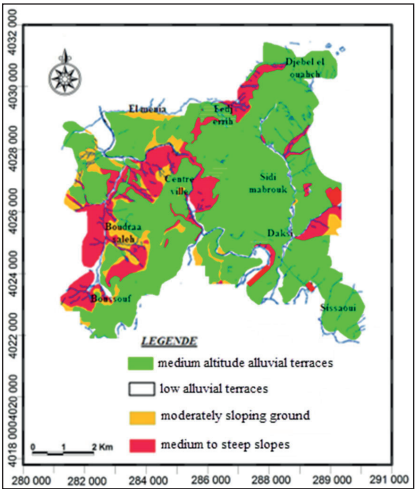


FIGURE 7. Slopes in Constantine
Source: own work.

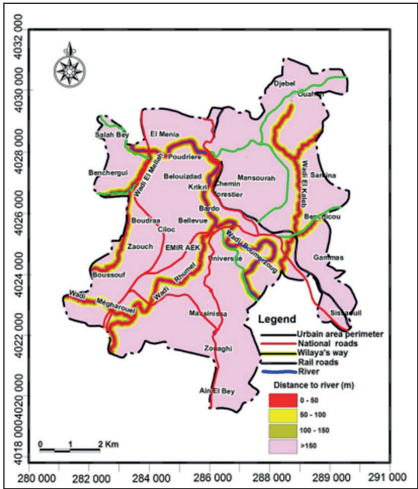


FIGURE 8. River influence on slope stability
Source: Bourenane et al. (2015).

TABLE 1. Classification according to the slope of the land parameter

Index (SLI ₁) = 1	Index (SLI ₂) = 2	Index (SLI ₃) = 3
slope <15%	slope between 15% and 45%	slope >45%
low risk	medium risk	high risk
green	orange	red

Source: own work.

TABLE 2. Classification according to the distance from watercourses parameter

Index (HI ₁) = 1	Index (HI ₂) = 2	Index (HI ₃) = 3
distance from watercourses more than 100 m	distance from watercourses between 50–100 m	distance from watercourses less than 50 m
low risk	medium risk	high risk
green	orange	red

Source: own work.

The rainfall factor. The climatic behavior of the studied region and the meteorological determinism show intense, time-concentrated rainfall, but in variable quantities and occurrences (Table 3). A commonly accepted danger and alert threshold by the international community is 50 mm per 24 hours (Guzzetti et al., 2007). The most at-risk rainy periods always require particular monitoring. Figure 9 shows the spatial distribution of the average annual rainfall over a 32-year period in the city of Constantine.

TABLE 3. Classification according to the rainfall parameter

Index (RI ₁) = 1	Index (RI ₂) = 2	Index (RI ₃) = 3
rainfall intensity < 50 mm	rainfall intensity between 50 and 100 mm / 24 hours	rainfall intensity > 100 mm / 24 hours
low risk	medium risk	high risk
green	orange	red

Source: own work.

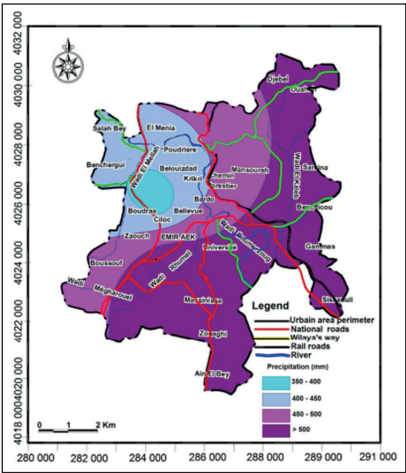


FIGURE 9. Spatial distribution of the average annual rainfall in 1980–2012 in the city of Constantine
Source: Bourenane et al. (2014).

Moderate but persistent rainfall proves to be more dangerous for the stability of a slope than short-duration heavy showers. The former promotes increased infiltration and deep soil saturation, leading to changes in soil properties and encouraging landslides, while the latter is mostly evacuated through surface runoff, having little effect on the water content of deep soil layers but causing significant erosion along the banks of the ouadis.

According to data obtained from the National Meteorological Office, two seasons dominate:

1. From January to March, it corresponds to winter and early spring.
2. From October to December, coinciding with autumn and early winter. However, two exceptional years show a concentration of rainfall in spring and early summer, specifically between April and June.

The soil nature factor. The soil type is represented by the soil index (SI) (Table 4). This index enables the development of a soil factor map (Fig. 10), as soil quality plays a crucial role in slope stability. Consequently, bedrock is preferred over artificial fill or loose, unconsolidated soil.

TABLE 4. Classification according to soil type

Degree of risk of indices (SI)	SI ₁ = 1	SI ₂ = 2	SI ₃ = 3
Ground instability	low	medium	high
Soil quality for construction	good	medium	bad
Color	green	orange	red

Source: own work.

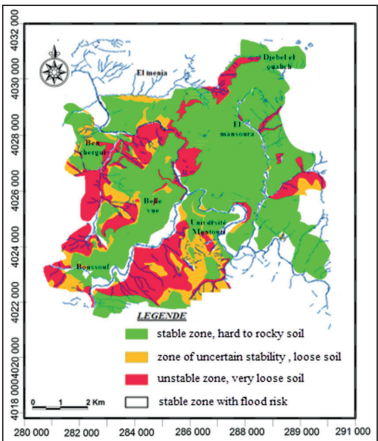


FIGURE 10. Types of soil in Constantine

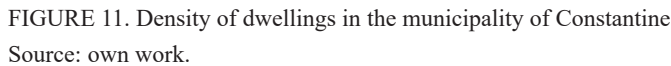
Source: own work.

The entropic factor: construction density index (CDI). Considering habitat density as a factor influencing the frequency of risk, the construction density index (CDI) has been assigned by category, as shown in Table 5 and Figure 11.

TABLE 5. Classification of the housing density index in the city of Constantine

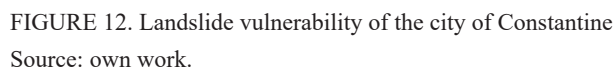
Index (CDI) dwelling	(CDI ₁) = 1	(CDI ₂) = 2	(CDI ₃) = 3
Number of homes per 1 ha	50–100	100–200	>200
Degree of risk	medium	high	very high
Color of risk	yellow	orange	red

Source: own work.



The degree of vulnerability is determined through a calculation in which we assessed the importance of each index for the maps. The overlay of these maps, all at the same scale, allows us to classify vulnerability into three descending levels as follows:

- Figure 12 can serve as an important tool to identify areas where urban improvement is necessary, as well as to optimize rescue operations in the event of a landslide.



Model validation is a crucial step in assessing its reliability and robustness. In the context of the study on the vulnerability of the city of Constantine to landslides, a comparative approach was adopted to compare the results of the developed model (Bui et al., 2020; Merghadi et al., 2020) with those from a previous reference study conducted by the consulting firm ARCADIS-Simecsol through its subsidiary EEG (Engineering Studies Company), and the Department of Urban Planning and Construction (DUC), in collaboration with the National Technical Control and Construction Agency (CTC) between 2001 and 2004.

These parameters were prioritized through a survey conducted among the scientific community, which allowed for weighting each factor according to its perceived importance. The weighting derived from this survey was then used to generate a vulnerability map at the municipal scale.

Comparison with the ARCADIS-Simecsol study

To validate the obtained map, a spatial overlay was performed with the one derived from the ARCADIS-Simecsol study, which serves as a reference for natural risk analysis in Constantine. This overlay allowed for a comparison of the areas identified as at risk in both studies. The results of this comparison revealed a similarity of approximately 97% between the two maps, indicating a strong alignment between the methods used. This proximity can be explained by the consideration of the same major terrain parameters, although the methodological approaches differ (multi-criteria modeling versus more in-depth geotechnical and historical analysis in the ARCADIS-Simecsol study).

Interpretation of results

The high similarity rate obtained (97%) demonstrates that the developed model is robust and relevant, particularly for urban-scale applications. This cross-validation also confirms the spatial consistency of the identified risk areas, whether they are regions known for their instability (such as the slopes of Bardo, Sidi M'Cid, or Djebel Ouahch) or new areas revealed through modeling.

The comparison with the results of the ARCADIS-Simecsol study provides evidence of the validity of the model used in our study. It confirms that, despite the use of distinct methods, both approaches lead to nearly identical conclusions. This validation grants the model scientific credibility and operational utility for urban planning, risk prevention, and natural disaster management in the city of Constantine.

The results show that areas with high slope gradients, weak geological formations, and intense or poorly managed urban development – particularly near drainage networks – correspond to zones of high vulnerability. Rainfall emerged as the most significant factor according to experts' opinions, reflecting the triggering role of water saturation in slope failure. This is consistent with global and regional literature (Guzzetti et al., 2007; Papathoma-Köhle et al., 2017), which emphasizes precipitation as the primary external driver of landslides.

The model also revealed areas of medium and low vulnerability, often corresponding to regions with gentle topography, stable lithology, and lower anthropogenic pressure. These spatial variations underline the importance of using a weighted multi-parametric approach rather than treating vulnerability as a uniform condition across the urban landscape.

Conclusions

The assessment of landslide vulnerability in an urban context, such as that of the city of Constantine, represents a major challenge for natural risk management. The systemic and multi-criteria approach adopted in this study allowed the prioritization of influencing factors and the generation of a precise map of risk areas. The results obtained, validated by a rigorous comparison with a reference study, confirm the relevance of the model used. This approach thus provides a solid scientific basis for guiding urban planning decisions, improving the area's resilience to natural disasters, and optimizing prevention strategies. By highlighting the importance of integrating physical, climatic, and human data, this study demonstrates the need for interdisciplinary approaches to address the challenges posed by environmental risks in urbanized areas.

This study addresses a scientific gap in the application of structured, expert-informed vulnerability modeling to complex urban environments in North Africa. The integration of physical, climatic, and human factors offers a holistic perspective on landslide risk that can guide both academic research and policy interventions.

From a practical standpoint, the model can support municipal authorities in prioritizing areas for risk mitigation, infrastructure reinforcement, and emergency planning. Furthermore, its adaptability makes it suitable for replication in other urban contexts with similar geological and climatic profiles.

While the model provides reliable results, it is based primarily on static spatial data and expert judgment. Future work could enhance accuracy by incorporating dynamic environmental data (e.g., real-time rainfall or soil moisture) and machine learning techniques to refine weightings. Moreover, including socio-economic vulnerability indicators would offer a more comprehensive risk assessment.

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Summary

A new approach for assessing landslide vulnerability at the urban scale: the case of the city of Constantine, Algeria. The natural risk of landslides is one of the most recurrent major natural risks in the world and one of the main concerns of civil security, since it causes a great deal of material and human damage. In Algeria, the civil authorities give great importance to risk management by implementing a natural disaster strategy that encourages the creation of prevention tools involving researchers, decision-makers, the general public, and the various actors in the field, which has led to studies on vulnerability to various natural phenomena. Constantine is one of Algeria's cities most at risk from landslides, due to its geology and the economic and social issues at stake. Therefore, we have developed a systemic, multi-criteria approach that puts into perspective the results of research on this risk in the city of Constantine, while defining the interactionist methodological model for vulnerability assessment and outlining the research context and problem. This research contributes to the field by proposing a reproducible, expert-informed methodology for urban-scale landslide vulnerability assessment. The approach is not only scientifically rigorous but also tailored for application in Algerian urban settings, offering new insights into the integration of local knowledge with spatial modeling techniques. The results provide a decision-support tool for urban planners, risk managers, and civil protection authorities.

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Non-linear seismic response for liquefaction analysis: pore water pressure and strain ratio approach

Keywords: liquefaction, DEEPSOIL, non-linear, Yogyakarta, strain ratio

Introduction

Earthquakes are natural disasters that frequently occur in Indonesia. As a country with a high possibility of an earthquake, the research to prevent infrastructure from seismic damage was reviewed, such as the microtremor analysis for seismic site effects (Fitri et al., 2018), the examination of shear strain for seismic micro-zonation of areas (Fitri, 2023) and the calculation for estimating the loss at locations (Purwana et al., 2022). Another natural problem induced by this issue is soil liquefaction. In 2006, soil liquefaction triggered by earthquakes caused massive damage in Yogyakarta (Sarah & Soebowo, 2013), initial liquefaction in Lombok (Tsimopoulou et al., 2020), and flow slides in Palu, both in 2018 (Kusumawardani et al., 2021).

The analysis of soil liquefaction is examined widely, especially in several locations in Yogyakarta. Commonly used methods in liquefaction

studies include systematic reviews (Fitri & Sawada, 2024), calculation of the safety factor and liquefaction potential index to assess the liquefaction potential (Wicaksono et al., 2023), integration with the liquefaction severity index (Dwiyantoro et al., 2023a), comparison of element simulations with simplified methods (Fitri & Sawada, 2025), and application of probabilistic methods such as analytic hierarchy process and fuzzy sets (Ilham & Putra, 2024). Another method is a non-linear analysis for seismic calculation; however, this approach is not widely used for liquefaction in Yogyakarta, although several studies applied this framework for liquefaction in other locations in Indonesia, such as liquefaction in Palu (Jalil et al., 2021b). In addition, there is vast potential for using non-linear analysis to assess liquefaction, such as pore water pressure (PWP) generation. However, only a limited number of studies combine strain calculation with PWP to assess liquefaction potential.

Although the PWP ratio is a general assessment of non-linear analysis in liquefaction potential, indicating that effective stress reaches zero, the shear strain ratio plays a crucial role in this examination. The amplitude of shear strain influences liquefaction resistance, as soil liquefaction resistance decreases as the amplitude of shear strain increases. Shear strain is a critical parameter in estimating the excess pore water pressure ratio during liquefaction, and it is an essential input for predicting pore water pressure. It affects pre-shearing as liquefaction resistance can be enhanced by small shear strain amplitudes (below 0.35%), while it can be decreased by larger amplitudes (above 0.6%) (Jas & Jana, 2024). Combining the PWP ratio and shear strain observation is important for practical stress assessment in soil liquefaction.

This research aims to generate the liquefaction analysis based on PWP generation and dissipation ratio correlated by the shear strain ratio in the underpass at the Yogyakarta International Airport (YIA). This research is expected to provide a comprehensive assessment of non-linear seismic response concerning liquefaction potential that offers a broad perspective of soil liquefaction methods.

Deepsoil program for liquefaction analysis

The primary cause of soil liquefaction is the accumulation of excess pore water pressure, referred to as the liquefaction mechanism. The pore water pressure can increase quickly during earthquakes or other cyclic loading events, possibly reaching a level where soil particles are forced apart, resulting in soil strength and liquefaction loss. The pore water pressure generation and dissipation

characteristics are integrated into DEEPSOIL's accurate modeling, which enables more realistic representations of soil behavior during seismic events. Soil models are dependent on pressure, and the software employs pressure-dependent hyperbolic models, which account for the influence of pore water pressure on soil behavior. DEEPSOIL provides a variety of pore water pressure generation models, including the Vucetic and Dobry theory for sands and the Matasovic approach for clays, allowing users to select the most suitable model for the specific soil characteristics (DEEPSOIL, 2024). The quantitative analysis in the software enables the determination of the pore water pressure ratio (r_u), which is crucial for determining the likelihood and severity of liquefaction. For instance, liquefaction is generally assumed to have occurred when r_u surpasses 0.8. Time-dependent analysis in DEEPSOIL allows users to observe the evolution of pore water pressure over time during a seismic event, thereby elucidating the development of liquefaction potential (Mei et al., 2015; Oktarina et al., 2023).

The approach of pore water pressure in DEEPSOIL was assessed by Mei et al. (2020), who validated the GQ/H+u model's capacity for generating cyclic soil behavior through dynamic centrifuge experiments with liquefiable sands. Another observation is a calibrated model in DEEPSOIL to a reasonable level of pore water pressure generation and an existing probabilistic possibility of a liquefaction curve capable of simulating strain-softening in the Marina earthquake simulation (Mojtahedzadeh & Siddharthan, 2021). Moreover, the efficiency of the simplified method for calibrating the pore pressure energy-based estimation model of Berrill and Davis is demonstrated by the excellent integration between the experiments of the excess pore pressure time histories and the empirical findings of the centrifuge tests, as well as the generated excess pore pressure descriptions of three case histories compared to the results accomplished by using another 1D non-linear code in DEEPSOIL (Mele et al., 2023).

In Indonesia, DEEPSOIL is widely used for seismic analysis. This software is applied to check the sensitivity for input motion; results indicate that the less amplification developed, the higher the natural frequency at the site (Syahbana et al., 2019). In addition, this program generates site-specific response analysis, which was used to generate the peak ground acceleration number at each depth and then employed to assess the liquefaction potential. DEEPSOIL is also used to extract the site-specific surface response spectrum from the equivalent-linear and non-linear analyses in Yogyakarta (Dwiyantoro et al., 2023b). Jalil et al. (2021a) also utilized this software to investigate the impact of local soil on the one-dimensional linear site response in the Balaroa, Petobo, and Jono Oge regions, considering the data from various sites within those regions.

This study analyzes liquefaction in an infrastructure project in Yogyakarta, based on the pore water pressure while also accounting for shear strain development in one-dimensional non-linear seismic analysis. This combined analysis is predicted to be a comprehensive assessment of the DEEPSOIL application in Indonesia. A complete understanding of the liquefaction mechanism from seismic analysis offers a better estimation of liquefaction potential and the effects of infrastructure damage due to earthquake-induced liquefaction.

Material and methods

Research location

This research location is the underpass access for the YIA project; the three boreholes conducted are named BH1, BH2, and BH3. All detailed locations are presented in Figure 1.

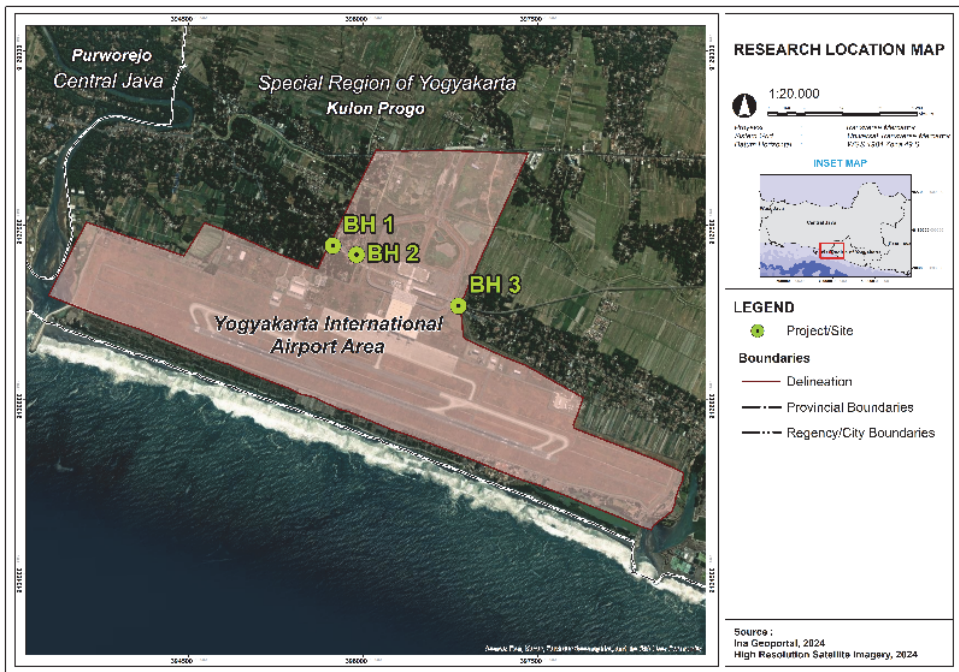


FIGURE 1. Borehole point in the project location
Source: own work.

This location represents a public infrastructure project that should be considered for more comprehensive liquefaction analysis as a vital building for the public. Moreover, based on the Indonesian liquefaction map (Ministry of Energy and Mineral Resources of the Republic of Indonesia [Kementerian ESDM], 2019), this area has a high possibility for earthquake-induced liquefaction. Detailed analysis is important for informing the stakeholders about infrastructure failure prevention.

Soil parameters

The three boreholes from the investigation site for the standard penetration test are simultaneously described as penetration numbers called N-values (NSPT). The soil subgrade, with a loose and medium consistency, is dominated by sand. The depth of the ground is around 10–12 m. The general information for BH1, BH2, and BH3 is depicted in Figures 2, 3, and 4, respectively.

Based on Figure 2, all the layers are dominated by sand and are considered to have a high possibility of liquefaction. However, in Figures 3 and 4, the presence of clay layers will cause a complex calculation for liquefaction analysis through non-linear analysis. In the empirical calculation for the assessment factor of safety, the clay layer is excluded from the calculation and is directly determined as the non-liquefiable soil, with no connection between the layered soil. Besides, the effects of liquefaction may not reach the surface if a shallow sediment layer overlies a non-liquefiable clay layer. The propagation of seismic vibrations and the overall susceptibility of the site to liquefaction may be influenced by the presence of clay strata, which can result in a more complicated behavior in the soil profile (Geotechnical Engineering Bureau & New York State Department of Transportation, 2007; Marasini & Okamura, 2014).

Three standard penetration test values are calculated to generate shear wave velocity (V_s), which could be calculated as follows:

$$V_s = 90.75 N_{60}^{0.304}, \quad (1)$$

where: N_{60} is the standard penetration testing value with correction.

The value is applied to plot the soil properties in DEEPSOIL (Hashash, 2024). This program is a non-linear site response analysis to simulate the liquefaction behavior of the layered ground. The constitutive model applied in this research is generalized/hyperbolic for developing the pore water pressure ratio (GQ/H+PWP) (Mei, 2018). In addition, the Darendeli model is utilized to generate the damping ratio value

and modulus reduction at every soil layer to produce an efficient hyperbolic depiction and offer greater precision at high strains (Darendeli, 2001). The shear strength calculated is adopted by the Mohr–Coulomb characteristic, in which the soil properties are directly calculated based on the consolidated overconsolidation ratio (OCR).

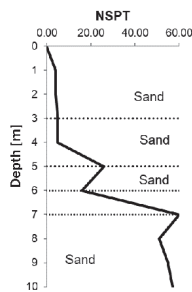


FIGURE 2. Detailed standard penetration test results (NSPT) for borehole BH1
Source: own work.

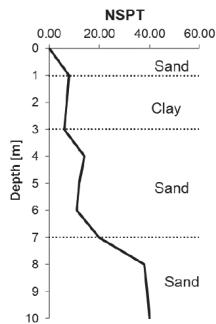


FIGURE 3. Detailed standard penetration test results (NSPT) for borehole BH2
Source: own work.

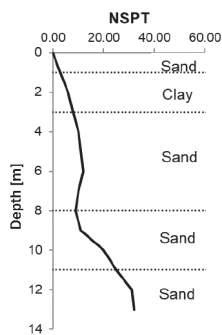


FIGURE 4. Detailed standard penetration test results (NSPT) for borehole BH3
Source: own work.

The PWP ratio (r_u) can be employed to depict the growth of PWP throughout loading cycles. Depending on the soil type, the DEEPSOIL numerical equipment illustrates variations in r_u using various implemented models. The application of Terzaghi's 1D consolidation theory in DEEPSOIL addresses the loss of excess PWP. Regarding the evaluation of PWP dissipation, this theory requires an understanding of the soil's coefficient of consolidation (C_v). The C_v is calculated for every layer of the specified site profiles. In general, the r_u is calculated as Equation 2, but in the DEEPSOIL non-linear analysis, the detailed calculation of r_u is presented in Equation 3.

$$r_u = \frac{\Delta u}{\sigma'_{vo}}, \quad (2)$$

$$r_u = \frac{pfNF(\gamma_c - \gamma_{tv})^s}{1 + pfNF(\gamma_c - \gamma_{tv})^s}, \quad (3)$$

where: PWP ratio is denoted by r_u , which generally develops as the change between PWP and effective vertical stress (σ'_{vo}), while N is the cyclic loading number, and γ_c is the cyclic shear strain rate. A one- or two-directional loading is indicated by the variable f , which can take on values of 1 or 2. The model parameters are p , F , and s , while γ_{tv} corresponds to the cyclic volumetric threshold of shear strain. The parameters p , F , and s of the PWP model were assessed in the present investigation using (Oktarina et al., 2023). Typically, the γ_{tv} for many sand soils is within the range of 0.01–0.02% (Dobry et al., 1982; Mei et al., 2018).

Traditionally, liquefaction is assumed to occur when r_u equals one, which signifies the zero value of the effective vertical stress in the sand. Nevertheless, laboratory simulation and field studies have shown that significant deformations can still occur in soils that are highly susceptible to liquefaction, even at lower r_u values. For example, pore pressure detectors produced direct instrumental evidence of liquefaction during the Superstition Hills earthquake in 1987 with Mw 6.6. During the earthquake, four transducers installed in a silty sand layer reported a r_u that exceeded 0.8, suggesting that liquefaction had taken place at the site (Bullock et al., 2021; Ko & Kayen, 2024).

In this research, to assess the liquefaction of the ground layer, the characteristics have been adopted in Table 1.

TABLE 1. Liquefaction based on r_u and shear strain [%]

Definition	Parameter
Full liquefaction	$0.8 \leq r_u \leq 0.9$ and $\gamma_{\max} \geq \gamma_{\lim}$ or $r_u \geq 0.9$
Marginal liquefaction	$r_u \geq 0.8$ and $\gamma_{\max} \leq \gamma_{\lim}$
No liquefaction	$r_u \geq 0.8$

Source: Raza and Ahmad (2024).

This study applied the ground motion of the Kobe earthquake, available in DEEPSOIL. The ground motion represents the seismic wave to simulate the dynamic behavior of the soil layer. Another Indonesian liquefaction case also used the Kobe earthquake and successfully calculated the non-linear ground response analysis (Jalil et al., 2021b).

Results and discussion

The shear wave velocity (V_s) value was initially calculated by correlation. However, for the simplification process, the soil layer is divided by consistency and soil properties. The final dynamic properties, such as shear wave velocity based on the calculation from the standard penetration test from BH1, BH2, and BH3, are presented in Figures 5, 6, and 7, respectively.

The results of the PWP ratio from DEEPSOIL for each layer are shown in Figure 8. Figure 8 also shows the variation value between BH1, BH2, and BH3. The initial r_u from a depth of 0–2 m is under 0.8 because the groundwater level (GWL) reaches a depth of 3 m. After the location of the GWL (–3.0 m), the r_u reaches more than 0.8 in BH1, whereas BH2 and BH3 are placed in that position at around a depth of 6 m. From three data boreholes, the maximum value of r_u in BH1 is approximately 4 m, BH2 is around 6 m, and BH3 is about 3 m in depth.

Generally, the impact of ground motions proved significant, as r_u values in susceptible soil strata consistently exceeded 0.8 under the GWL. This observation underscores the significance of depth-specific evaluations in assessing liquefaction potential. Furthermore, the findings indicated that the soil layer exacerbated susceptibility to soil liquefaction. It has been proven that the r_u value did not reach 0.8 in the clay layer; the r_u value in BH2 and BH3 will reach 0.8 in sandy conditions. Moreover, engineers must consider the liquefaction potential to design substructures in this layer, such as foundations or tunnels. The value indicating

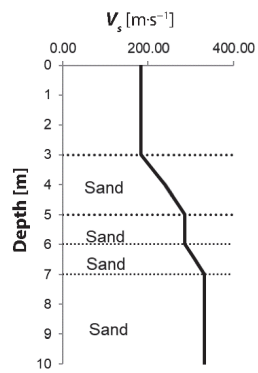


FIGURE 5. Shear wave velocity (V_s) for borehole BH1
Source: own work.

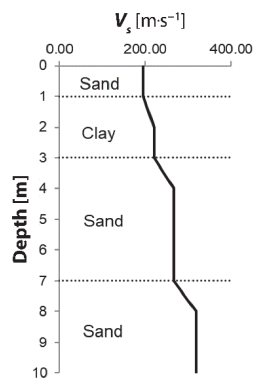


FIGURE 6. Shear wave velocity (V_s) for borehole BH2
Source: own work.

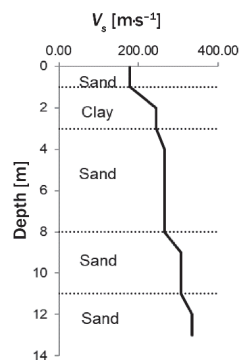


FIGURE 7. Shear wave velocity (V_s) for borehole BH3
Source: own work.

the soil’s liquidity (r_u more than 0.8) also induces another problem, such as a decrease in shear strength and landslides. Although the landslide analysis also must consider complex methodologies such as the finite element method or the limit equilibrium method (Fitri & Wahyuni, 2022), this study is limited to the liquefaction behavior investigation only.

The liquefaction analysis for the non-linear approach is not only the r_u value but also the strain ratio; the result of strain from DEEPSOIL is shown in Figure 9. Based on this data, the strain ratios almost correlate with the r_u value. The maximum strain rate is BH1 with around 8% at a depth of 3 m, while BH2 and BH3 reach the maximum of around 4% at about a depth of 6 m. The maximum shear strain ratio values have a correlation with the maximum r_u value in the same depth and boreholes.

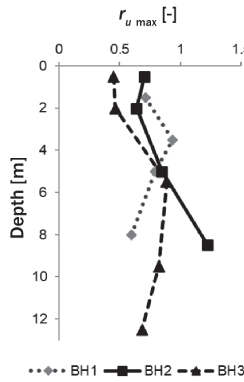


FIGURE 8. Result of pore water pressure ratio (r_u) from boreholes BH1, BH2, and BH3 generated by DEEPSOIL

Source: own work.

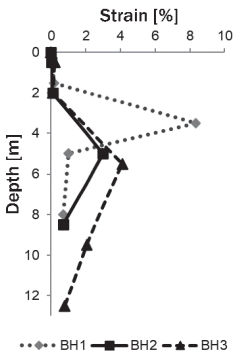


FIGURE 9. Strain test results from boreholes BH1, BH2, and BH3

Source: own work.

According to the result of r_u and strain for each depth, the soil layer represents the combined effect of shear strain, and r_u is depicted against time in Figures 10, 11, and 12, respectively. The combined output is based on the maximum value of r_u and the full liquefaction condition.

The general combination results show different outputs for each borehole; this is consistent with Raza's and Ahmad's findings (2024), which indicate that the input ground motions resulted in the generation of r_u and significant shear strain values, with both parameters demonstrating the rising and falling trends of each layer. Therefore, the soil liquefaction was determined at various depths.

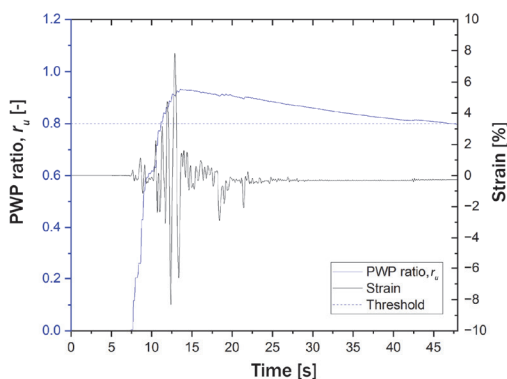


FIGURE 10. Correlation between strain and pore water pressure ratio (r_u) in borehole BH1 at the maximum r_u layer

Source: own work.

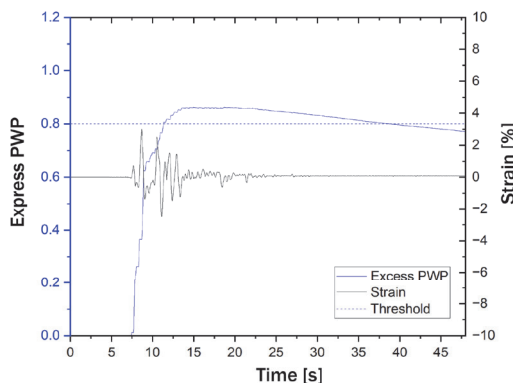


FIGURE 11. Correlation between strain and pore water pressure ratio (r_u) in borehole BH2 at the maximum r_u layer

Source: own work.

The cyclic loading of soil induces shear strains and increases pore water pressure during an earthquake. Throughout ground shaking, the development of excess pore water pressure and shear strain undergoes a time-dependent analysis. In order to more accurately represent the cyclic demand on soil, advanced methods consider the intensity, duration, and frequency content of input motions (Kramer & Mayfield, 2007). The empirical method typically applies peak ground acceleration and magnitude scaling factor from a particular earthquake magnitude, which are dependent on the magnitude (M) and distance (R) of the earthquake, to determine the seismic aspect of the liquefaction analysis. This approach yields more reliability and complexity in field conditions.

Based on the results of the r_u maximum value and strain ratio approach, the output emphasizes the deformation behavior of the ground, presenting a maximum value around 8 seconds for BH2 and BH3, and about 13 seconds in BH1. The peak of the r_u value also differs among the three boreholes. The highest strain ratio value does not always correspond to the highest value of r_u . In addition, this condition means that the strain value indicates the beginning of deformation, although the liquefaction condition is not in the full liquefaction situation. As the shear strain accumulates, it will contribute to the pore water pressure build-up and lead to soil failure.

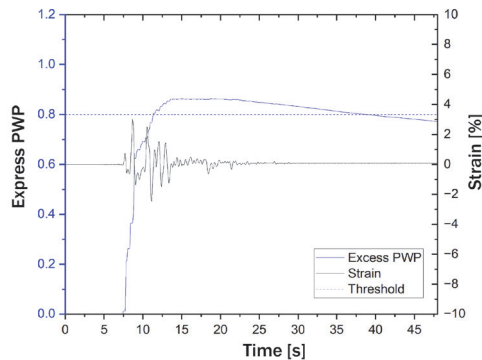


FIGURE 12. Correlation between strain and pore water pressure ratio (r_u) in borehole BH3 at the maximum r_u layer

Source: own work.

Generally, the assessment for soil liquefaction in Indonesia is limited to the simplified method. This method examines whether the site has high liquefaction potential or not. However, this study's non-linear ground response approach simulates the real phenomena of the ground based on the time history

that yields the dynamic properties in detail. The specific properties of the seismic ground response are expected to precisely predict the soil liquefaction phenomenon rather than depend on the simplified method.

Conclusions

Generally, the variety of soil properties in different boreholes offers diverse shear wave velocities with the same consistency of sand. This circumstance leads to the result of non-linear seismic analysis at each depth. The maximum pore water pressure ratio (r_u) value is at different depths. The BH1 shows maximum value in depth around 3 m, and BH2 and BH3 at about 6 m. The clay layer affects the r_u and the correlation with the time in ground motion analysis. Moreover, all the boreholes show that $r_u > 0.8$.

Regarding the other dynamic examinations, such as shear strain, the shear strain ratio, which corresponds with the same depth of r_u value, shows around 8%, 4%, and 4% for BH1, BH2, and BH3, respectively. The maximum r_u value is strongly correlated with the highest shear strain ratio. However, the value does not correspond with the r_u at a particular ground motion time. The maximum point between the r_u and the shear strain ratio is reached in different periods. Therefore, based on the correlation between r_u and strain ratio, the three boreholes predict a full liquefaction condition from non-linear seismic response analysis. The output proves that the strong behavior of the strain ratio and r_u systematically contributes to the liquefaction condition of the ground. Engineers should take into account the dynamic properties of the ground carefully.

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Summary

Non-linear seismic response for liquefaction analysis: pore water pressure and strain ratio approach. The liquefaction analysis in infrastructure projects is crucial as a preventive action for damage, especially in Indonesia, an earthquake-prone country. A wide range of liquefaction analyses is usually applied in Indonesia, such as non-linear analysis to generate the pore water pressure ratio (r_u) to assess the potential of liquefaction. However, this approach does not fully present the complex behavior of layered soil in the soil-effective stress framework. The shear strain ratio is important for offering a comprehensive understanding of soil liquefaction analysis. This research aims to examine the combination of r_u and shear strain ratio to assess liquefaction in layered ground. The DEEPSOIL program is used to generate the dynamic properties in each layer. The results show a strong correlation between the r_u and strain ratio in each depth and borehole. However, they have diverse values in connection with the period of the ground motion approach. As a result, the evaluation and combining approach in the non-linear seismic response analysis of the three boreholes indicates a full liquefaction condition at a particular depth.

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Methodology for estimating waste generation and cost implications in panel building demolition

Keywords: cost estimation, construction, methodology, panel, prefabricated buildings, waste

Introduction

The extensive construction of prefabricated buildings during the latter half of the 20th century constitutes a substantial proportion of the housing stock (Barnaś et al., 2023) in several European countries, including the Czech Republic, Bulgaria, and Poland. In recent decades, significant investments have been directed towards the regeneration of these structures, such as the replacement of outdated windows with modern alternatives possessing better thermal properties, and the application of External Thermal Insulation Composite Systems (ETICS) (Turk et al., 2022; Kubečková, 2023) to prolong their service

life and enhance their resilience to climate change (Jakeš, 2023). Despite these interventions, it is anticipated that, over time, the need to demolish or deconstruct these prefabricated buildings (also referred to as panel housing estates) will become inevitable. In Poland, for instance, there has been an ongoing debate regarding whether these buildings should be demolished or their lifespan further extended through refurbishment (Malazdrewicz et al., 2022). In the Czech Republic, a limited number of prefabricated multi-story buildings have already been demolished, either because they were never completely finished or as a result of severe tenant-induced damage. Consequently, it is imperative to develop timely strategies for the effective management of construction waste generated from the future demolition/deconstruction of such structures.

The academic literature defines construction and demolition waste (CDW) as waste produced throughout the entire lifecycle of a building, encompassing construction, repair, maintenance, demolition, and deconstruction activities (Gálvez-Martos et al., 2018). The generation of CDW poses a significant challenge to the construction industry, primarily due to its environmental impact (Kabirifar et al., 2020). Given the considerable proportion of CDW that is landfilled globally (Ajayi et al., 2016), a variety of management strategies have been proposed. These include, but are not limited to, the implementation of CDW management plans, economic instruments (such as landfill taxes to incentivize recycling), site waste management plans, design strategies aimed at minimizing waste, material efficiency practices, building deconstruction, and waste sorting (Gálvez-Martos et al., 2018).

All aforementioned measures align with the principles of the circular economy, which can be integrated from the earliest project stages, such as through green procurement practices (Górecki, 2020), throughout the operational phase via effective maintenance (Cerić et al., 2019; Kilić Pamuković et al., 2020), culminating in end-of-life strategies (Benachio et al., 2020) that prioritize waste separation (Sobotka et al., 2015) and the assessment of material reuse or recycling potential (Sanchez & Haas, 2018). The adoption of such environmentally sustainable approaches yields multiple benefits, including reduced landfill waste, economic gains such as job creation in the recycling sector, and societal advantages, for example, diminished landfill-related nuisances impacting local communities (Plebankiewicz et al., 2016; Androjić & Marović, 2019; Purchase et al., 2021; Bochko et al., 2024). Nevertheless, the construction sector retains significant potential for further waste reduction, as the majority of CDW is non-hazardous and suitable for recovery (Spišáková et al., 2021). In this context, Spišáková et al. (2021) advocate for the execution of a construction and demolition waste audit prior to the commencement of demolition/deconstruction activities.

This study is motivated by two principal factors. First, the volume of CDW generated is exceptionally high; according to the latest available statistics published by the statistical authority (Czech Statistical Office [CSU], 2022), CDW accounted for approximately 70% of total waste produced in the Czech Republic between 2017 and 2021.

Second, there is a notable absence of a suitable methodology for the quantification or prediction of separated CDW arising from the demolition of existing panel buildings. At present, estimates of CDW generated during demolition in the Czech Republic are typically based either on demolition budgets – which provide only aggregate waste quantities without further material separation – or on the subjective assessments of designers drawing on their practical experience. Therefore, the objective of this research is twofold: (a) to develop a methodology for estimating the quantitative recycling potential of waste generated from the demolition of panel buildings, and (b) to assess the associated cost implications within the context of the Czech Republic.

Material and methods

Prefabricated, or panel buildings are typically characterized as multi-story structures composed of factory-made panels for floors and walls, which are interconnected both vertically and horizontally to form a rigid, box-like structure (Malazdrewicz et al., 2022). The vertical wall panels typically span the full height of a single story (Shemie, 1973). In the Czech Republic, a variety of prefabricated construction systems have been implemented, including the GOS, VOS, and VPOS systems, among others (Kubečková & Vrbová, 2021); specific systems are often characteristic of particular regions. For the purposes of this study, the T06B panel system was selected due to its status as the most prevalent prefabricated building system in the Czech Republic. The T06B system itself comprises several regional variants. For analytical purposes, the T06B-KDU variant was chosen, which is predominantly found in the contemporary South Moravian Region, Vysočina Region, as well as the Zlín and Olomouc Regions (Panelaky.info, 2020).

In this study, we focus exclusively on the point-access block configuration of the T06B-KDU panel building system. For the purpose of quantifying CDW disposal costs associated with the demolition of a point-access panel building, the “cube” variant was selected as a representative case study. This building type is exemplified, for example, by structures located in the panel housing estate complex in Brno–Žabovřesky, as illustrated in Figure 1.



FIGURE 1. View of a point-access panel building

Source: own work.

Figure 2 reproduces an excerpt from the original project documentation and depicts a typical floor plan of the analyzed point-access building constructed using the T06B-KDU panel system. Although the graphic quality of the floor plan is relatively low, the authors hereby wish to point out the fact that the original documentation of these buildings is only in paper form of 50-year-old historical documents, the readability of which can be difficult and therefore can complicate CDW estimates.

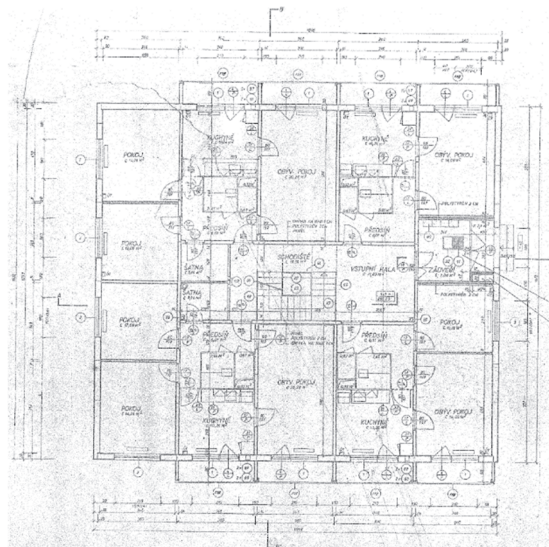


FIGURE 2. Typical floor of the surveyed point-access panel building

Source: kindly provided by SBD Průkopník (construction housing cooperative).

The data processing for this study was primarily based on the original project documentation and the associated historical bill of costs. For the analyzed structure, a list of all prefabricated panels, including their respective weights, was obtained from SBD Průkopník (1969). The weights of other structural components were estimated using the BUILDpower S budgeting software at a 2024 price level (RTS a.s., 2024).

The subsequent data analysis categorized the various types of CDW in accordance with the Czech Decree No. 8/2021 Coll. – the Waste Catalogue (Vyhláška č. 8/2021), which classifies CDW based on material characteristics and aligns with the European Waste Classification system EWC-STAT (European Union [EU], 2024). For clarity and auditability, the waste classification was conducted first by construction element and then by the corresponding waste code. Specifically, waste from foundation structures was subdivided into concrete (originating from the foundations), metal (from reinforcement), bitumen strips (from waterproofing layers), and plastic (from plastic sheeting).

Vertical structural components generate CDW comprising concrete (reinforced concrete panels), metals (reinforcement), bricks (brick cores and insulation liners), polystyrene with perlite used in External Thermal Insulation Composite Systems (ETICS), mixtures (internal plaster), and ceramics (ceramic tiles from bathrooms and toilets).

The horizontal structures are divided into three parts: specifically reinforced concrete structures (prefabricated panels), floor construction (floor composition), and roofing (roof composition). Horizontal panels, internal vertical panels, and staircase precast reinforced concrete panels are classified under reinforced concrete structures. Vertical reinforced concrete panels forming the envelope are classified under perimeter casing.

For floor constructions, the following CDW types are expected: concrete (screed), metals (reinforcement), ceramics (tiles), wood (wooden flooring), PVC, carpet materials, and polystyrene. Waste from doors and window fillings is categorized into wood and glass (internal doors), plastic and glass (facade window fillings), and iron and steel (metal garage doors). The roof structure waste consists primarily of aerated concrete insulation (slump layer), bitumen strips (waterproofing), and stones (pebbles).

The results of the analysis were compared with the current approach used for quantifying CDW in the preparation of demolition cost estimates (bill of costs for the demolition). This conventional approach primarily relies on the “demolition” item within the bill of costs, where the critical parameter is the proportional volume of structural components. Specifically, volumes of external walls, internal walls, horizontal load-bearing structures, and staircases are considered without subtracting openings. The volume of these structures is then divided by the total

built-up area of the building, which is derived from the adjacent landscaped terrain. Budgeting software typically estimates the total expected waste weight based on internal databases, but does not further disaggregate the waste, e.g., by type.

The analysis encompasses a comparative estimation of the costs associated with recycling separated CDW versus the disposal of mixed waste through landfilling. Cost data (charges) for both recycling and disposal processes were obtained from the RTS price database (RTS a.s., 2024).

Results and discussion

Table 1 presents the basic characteristics of the case study panel building previously depicted in Figure 1. The description includes the main dimensions and relevant technical specifications. The building is a residential structure comprising four above-ground floors and one subterranean level. Each residential floor contains four apartment units: two units of type 2 + 1 and two units of type 3 + 1. Here, the notation X + Y denotes the apartment configuration, where X (either 2 or 3) represents the number of rooms, and Y (equal to one) indicates the presence of a separate kitchen.

TABLE 1. Input data of a point-access panel building (T06B-KDU)

Specification	Value	Specification	Value
Number of stories	4 + 1	Length [m]	18.74
Number of tracts	5	Width [m]	16.40
Thermal insulation of the building (yes/no)	yes	Thickness of thermal insulation [m]	0.14
Paving area [m ²]	171.06	Built-up area of one story [m ²]	307.34
PVC area [m ²]	595.95	Built-up space [m ³]	4,598.65
Carpet area [m ²]	198.65	Facade area [m ²]	580.16
Parquet floor area [m ²]	256.80	Number of residential units per floor	4

Source: own work.

Table 2 presents the estimated quantities for all the structures included in the analysis. The data include total CDW volumes for the entire building as well as normalized CDW values expressed per 1 m³ of built-up area. The CDW is categorized into seven distinct waste categories, with each waste type specifying the feasibility of recycling based on consultation with recycling centers and landfill sites (Kovosteel recycling, s.r.o., and FCC Czech Republic, s.r.o.).

TABLE 2. Analysis results for a point-access panel building

Waste category	Quantity of waste		Recyclability [yes/no]
	per whole building [t]	per 1 m ³ built-up space [t·m ⁻³]	
Foundation construction of which	276.92978	0.06022	–
17 01 01 Concrete	272.16255	0.05918	yes
17 03 02 Bituminous mixtures	4.73030	0.00103	no
17 02 03 Plastic	0.03693	0.00001	no
Perimeter casing of which	362.18580	0.07876	–
17 01 01 Concrete	311.36000	0.06771	yes
17 01 01 Iron and steel (reinforcement)	1.65000	0.00036	yes
17 01 07 Mixtures of bricks	41.00656	0.00892	yes
17 06 04 Insulation materials	8.16924	0.00178	no
Reinforced concrete structures of which	1,060.44003	0.23060	–
17 01 01 Concrete	875.38800	0.19036	yes
17 01 01 Iron and steel (reinforcement)	27.55200	0.00599	yes
17 01 01 Mixtures (interior plaster)	157.50003	0.03425	yes
Floor construction of which	151.29183	0.03290	–
17 01 01 Concrete	136.30226	0.02964	yes
17 01 01 Iron and steel (reinforcement)	5.66373	0.00123	yes
17 01 07 Mixtures of tiles	3.88752	0.00085	yes
17 02 01 Wood	2.97250	0.00065	no
17 06 04 Pure polystyrene	0.12990	0.00003	no
20 03 07 PVC flooring	1.93612	0.00042	no
20 03 07 Carpets	0.39980	0.00009	no
Brick core of which	29.45596	0.00641	–
17 01 07 Mixtures of bricks	21.54065	0.00468	yes
17 01 07 Mixtures of tiles	7.91531	0.00172	yes
Facade fillings of which	7.87681	0.00171	–
17 09 04 Wood + glass	2.07200	0.00045	yes
17 09 04 Plastic + glass	2.31261	0.00050	yes
17 04 05 Iron and steel (doors)	3.49220	0.00076	yes
Roofing of which	25.11277	0.00546	–
17 03 02 Bituminous mixtures	3.00877	0.00065	no
17 05 04 Soils and stones	15.55025	0.00338	yes
17 01 02 Aerated concrete	6.55375	0.00143	no
Total amount of monitored waste	1,913.29298	0.41606	–

Source: own work.

Table 3 summarizes the total quantities of CDW classified by recyclability. The analysis indicates that up to 98% of the waste generated from the demolition of a prefabricated building has the potential for recycling. The remaining 2% is likely to be disposed of via landfilling or incineration. Consultations with recycling facilities revealed that, within the current Czech Republic waste management framework, certain materials – including bitumen strips, plastic sheeting, polystyrene, wooden flooring, PVC and carpet, as well as aerated concrete – are not recyclable.

TABLE 3. Recyclability of generated waste

Waste	Quantity	
	[t]	[%]
Recyclable	1,885.35567	98.54
Non-recyclable	27.93731	1.46

Source: own work.

The recyclability potential of CDW has significant cost implications. Waste that is eligible for recycling incurs lower disposal costs, whereas non-recyclable waste is associated with higher expenses (charges) due to its predominant disposal via landfilling or incineration. These cost differentials are detailed in Table 4 (converted to EUR with exchange rate 1 EUR = 25.185 CZK, valid to 31.12.2024).

According to the data presented in Table 4, the total estimated cost for the disposal of waste generated from the demolition of the case study panel building (comprising both recycling and landfilling) amounts to approximately EUR 29,000. In contrast, if the waste were not segregated and consequently required full landfilling, the disposal cost would be EUR 240.62 per 1 ton of mixed waste, resulting in a total expense of approximately EUR 460,375.44 for the entire volume of CDW. This reflects a cost differential of approximately EUR 431,000, which can be interpreted as the potential savings attainable through the full utilization of the current recycling capabilities available in the Czech Republic.

Furthermore, these findings were compared with waste quantity estimates derived from the demolition budget generated using the BUILDPower S software. The proportion of the structure in a point-access panel building was found to be approximately 19% of the built-up space, leading to the selection of the budget item “981014713R00 – Demolition of buildings by mechanization, reinforced concrete, construction up to 20%”. This item prescribes a predefined waste generation factor

of $0.42 \text{ t} \cdot \text{m}^{-3}$ of built-up area. Given the case study building's built-up volume of $4,598.65 \text{ m}^3$, the estimated waste quantity is 1,931.433 tons, which is marginally higher than the estimate obtained via the more detailed analysis proposed by the new methodology.

TABLE 4. Waste recycling/disposal costs, source of unit cost data

Type of waste	Unit waste recycling/ disposal costs [EUR/t]	Quantity [t]	Total costs [EUR]
17 01 01 Lightly reinforced concrete, piece size up to $1,600 \text{ cm}^2$	12.61	1,787.57857	22,535.49
17 01 02 Aerated concrete	20.13	6.55375	131.93
17 01 07 Mixed waste concrete, bricks, tiles, and ceramic products, piece size up to 1600 cm^2	18.48	74.35004	1,374.23
17 02 01 Wood	62.97	2.97250	187.19
17 02 03 Plastic	240.62	0.03693	8.89
20 03 07 PVC floor covering	240.62	1.93612	465.87
20 03 07 Carpets	240.62	0.39980	96.20
17 03 02 Bitumen strips	240.62	7.73907	1,862.17
17 05 04 Soils and stones	18.05	15.55025	280.63
17 06 04 Polystyrene	240.62	8.29914	1,996.93
17 09 04 Wood + glass	114.35	2.07200	236.94
17 09 04 Plastic + glass	114.35	2.31261	264.46
17 04 05 Iron scrap over 4 mm thick	-133.16	3.49220	-465.01
Total		1,913.29298	28,975.91

Source: RTS a.s. (2024).

Additionally, the weight of foundation structures must be considered, with an estimated volume of approximately 108 m^3 . Based on RTS a.s. data, the expected waste weight from the foundation structures is approximately 215.58 tons. Consequently, the total waste weight estimated from the demolition budget amounts to 2,147.013 tons, representing a discrepancy of roughly 10% compared to the detailed analysis. This difference may be attributed to the exclusion of certain structural components related to construction production, such as building services, locksmith structures, and plumbing installations.

Conclusions

The study introduces a methodology for the economic analysis of CDW management specific to panel buildings in the Czech Republic. This methodology addresses the existing gap in methodologies capable of accurately quantifying the volume of source-separated CDW generated during the demolition of panel buildings. Consequently, it enables the assessment of recycling potential and facilitates the evaluation of associated economic factors, considering both feasible recycling processes and the necessary landfilling.

The analysis yields several key conclusions. First, the results demonstrate that implementing sorting and recycling strategies for CDW can substantially reduce disposal costs compared to the alternative of disposing of the entire waste volume via landfilling. Second, the case study of a point-access panel building indicates that up to 98% of the generated CDW could be recycled based on the structural components considered. Third, the proposed methodology provides the capability to monitor waste quantities by individual material categories, thereby supporting the optimization of recycling capacities.

At a theoretical level, the analyses conducted contribute to an enhanced understanding of the management challenges associated with CDW from prefabricated residential buildings, which constitute a substantial portion of the housing stock in Central, Eastern, and Southeastern Europe. From a managerial perspective, the proposed methodology enables practitioners to effectively plan for waste separation by considering both the necessary technological solutions and the required processing capacities.

This research is not without limitations. The analysis did not evaluate the cost-effectiveness or operational feasibility for builders in separating individual CDW fractions. The primary objective was to develop a methodology for quantifying the volume of source-separated waste while maximizing recycling potential, thereby promoting environmental benefits through the reduction of landfill disposal. Additionally, the analysis excluded certain building components, such as sanitary installations, heating systems, and minor structural elements (e.g., staircase railings, plumbing fixtures), which may contribute to the overall waste generation.

The question remains to what extent technologies for separating and recycling CDW are available in the Czech Republic. Building on the proposed methodology, future research should assess the readiness of recycling facilities to accommodate increased future waste volumes and to process waste types that are presently non-recyclable. At present, demolition and deconstruction of panel buildings

in the Czech Republic remain relatively rare. Identification of a suitable panel building scheduled for demolition would provide an opportunity to validate and potentially refine the proposed methodology and its estimations of separated waste quantities.

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Summary

Methodology for estimating waste generation and cost implications in panel building demolition. This study addresses the impending challenge of construction and demolition waste (CDW) generation from the Czech Republic's extensive panel housing estates, constructed between the 1950s and 1990s. These structures, representing a significant portion of the national housing stock, will eventually reach their operational lifespan, necessitating systematic waste management strategies. A novel estimation methodology is proposed to quantify demolition waste volumes through material-specific decomposition of panel building structures. The T06B panel system, widely deployed in Czech housing estates, serves as a selected case study. The structure, according to the Waste Catalogue, is used for the classification of specific waste types. From a cost perspective, individual fees for waste disposal or recycling are taken from the budgeting program database. The proposed methodology facilitates the predictive modeling of both demolition waste quantities and associated financial expenditures for disposal/recycling of individual waste categories, such as concrete, bricks, iron, plastic, etc.

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A case study of complex pollution characteristics associated with suspended particulate matter and heavy metals

Keywords: sediment transport, water pollution, distribution coefficient, suspended particulate matter

Introduction

Aquatic ecosystems are regulated by dynamic interactions across three primary interfaces: the water column, suspended particulate matter (SPM), and bottom sediments. The SPM acts as a key agent in the transformation of substances, especially heavy metals (HMs). Moreover, the mobility and sorption properties of SPM require detailed study and evaluation

(Li et al., 2022). The SPM comprises both organic and inorganic matter suspended within the water column and is quantitatively expressed through total suspended solids (TSS), typically measured in $\text{mg}\cdot\text{L}^{-1}$. All natural water bodies contain SPM to varying degrees, with their concentrations and characteristics being vital determinants of ecological balance (Xia et al., 2022). Anthropogenic activities have notably increased SPM concentrations, thereby altering key physical, chemical, and biological parameters of freshwater systems. The environmental impact of SPM is influenced by multiple factors: its concentration, exposure duration, and physicochemical properties, including particle size and chemical composition (Qasim, 2021). The particle size of SPM directly influences its suspension time and depth distribution, as well as its metal-binding capacity. Due to their colloidal nature and high surface area, smaller particles demonstrate higher sorptive potential, particularly for dissolved HMs. Consequently, SPM significantly contributes to the redistribution of metal pollutants in riverine ecosystems.

The impact of SPM on watercourse living organisms is, as a rule, greater at higher concentrations. Furthermore, the impact of living organisms' exposure to concentrations of SPM is subject to variation according to period, duration of exposure, and the condition of the living organisms themselves. The chemical composition of SPM is also a critical factor because it controls its toxicity. The distribution of SPM particles by size is a critical factor in determining the duration of SPM suspension, as well as its distribution within the depth of the water reservoir and its sorption affinity (Fig. 1). SPM particles of a smaller size will be suspended for a greater period than particles of a larger size. SPM size has been demonstrated to exert a significant influence on sorption affinity. SPM particles, due to their colloidal nature, possess a high sorption capacity due to their small size. In the context of river water systems, SPM exhibits a high capacity for the sorption of dissolved HMs, attributable to its active nature and substantial surface area (Gantayat et al., 2023).

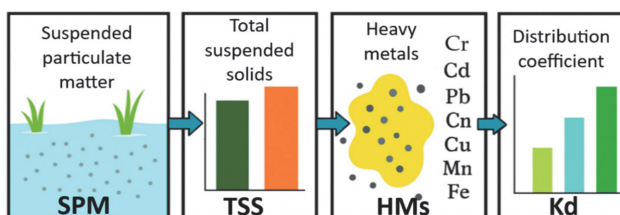


FIGURE 1. The distribution of heavy metals and the general scheme of this study

Source: compiled by the authors using AI graphical tools.

The distribution of HMs in river water systems can be affected by different control parameters, including pH, salinity, turbidity, and TSS. The use of correlation analysis facilitates the identification and prediction of the behavior of HMs (Kuznietsov & Biedunkova, 2025). The calculation of the distribution coefficient (K_d) facilitates the analysis of the distribution balance of HMs between the SPM and the liquid phase. The study by Kucuksezgin et al. (2007) demonstrates that variations in the K_d of the HMs between the SPM phase and the liquid phase are caused by water chemistry and TSS concentrations. Specifically, Kaiser (1989) also discovered that phytoplankton blooms enhance the amount of HMs in the SPM. The distribution of Cd, Cu, and Cr showed significant contamination in SPM, while Zn content showed a low degree of contamination. The transfer of free and bound forms of HMs to the SPM phase is associated with the formation of hydroxides that are insoluble in water, especially for Mn and Fe (Zhang et al., 2018). Nazeer et al. (2014) found that suspended sediment samples were more contaminated with Ni, Pb, and Cd compared to bottom sediment samples. In the study by Zeng et al. (2019), the occurrence of heavy metals had the following order of decreasing concentrations in SPM: Mn > Zn > V > Cr > As > Cu > Ni > Pb > Cd. According to the results of another study, the following order of enrichment of SPM with heavy metals was observed: Zn > Pb > Cr > Cd > Cu > Ni. At the same time, the distribution coefficient values of metals between solid particles and dissolved fractions had the following order: Pb > Cr > Zn > Cu > Cd > Ni (Zheng et al., 2013). Distinct distribution values obtained for disparate water bodies thus necessitate separate studies, as each possesses unique dynamics and hydrochemical parameters.

Among the various pollutants, HMs adsorbed by sediments are of particular interest due to their mobility and toxicity within the water ecosystem (Pourabadehei & Mulligan, 2016; Singovszka & Balintova, 2020). The deposition process and content of HMs in reservoir sediments have been extensively studied, but the distribution of HMs in explosives has been considered much less frequently. The scientific problem lies in the lack of quantitatively verified relationships between TSS/SPM particle size distribution, K_d partition coefficient, and the phase content of HMs for conditions of medium-sized rivers of the temperate zone; the practical problem lies in the lack of local thresholds and reference parameters (seasonal K_d by particle classes, event-based turbidity triggers), without which it is not easy to model HM mobility and plan effective water protection measures. SPM is an essential carrier of HMs in a water body, and the evaluation of HM contaminations in SPM is critical as a preliminary step towards effective management of surface water and a further base for water

resource sustainability (Zeng et al., 2020; Idan et al., 2021; Wang et al., 2025). However, this is not a common occurrence in developing countries. The purpose of this research endeavor was to undertake a comprehensive examination of the TSS concentrations and SPM particles size distribution, with a view to identifying the distribution of HMs (Cd, Pb, Cr, Zn, Cu, Mn, Fe) between SPM and water in the Styr River (Fig. 1). The primary objectives are as follows: first, to investigate the dynamics of TSS concentration; second, to investigate the distribution of SPM particles by size; third, to investigate the relationship between TSS and HM concentrations; fourth, to investigate the dynamics of HM content in water and SPM; and fifth, to estimate the distribution of HMs in dissolved (water) and particulate (SPM) phases.

Thus, our study aims to fill these gaps for the Styr River through consistent measurement of TSS, SPM size composition, HM phase content, and the calculation of local K_d , increasing the applied value of monitoring and management decisions.

Material and methods

The Styr River water samples were collected monthly during the cold (January–March, October–December) and warm (April–September) periods, during 2024. The Styr River is in north-western Ukraine with a length of 483 km, a drainage basin of 13,130 km², and an annual mean flow rate of 69 m³·s⁻¹ (Stelmakh, 2024). Samples were taken at a point 183 km from the mouth of the Styr River (51°21'53.2" N, 25°51'01.3" E). The territory within this riverbed section belongs to the Polissya zone of Ukraine. The administrative location is the city of Varash, Rivne region, at the water intake site of the Rivne Nuclear Power Plant. Up to this point, the river is affected by agricultural, domestic, and industrial wastewater.

Figure 2 presents an overview of the study's workflow. An amount of 1 L of water was filtered through blue strip filter paper to determine TSS using standard gravimetric procedures. The retained SPM on the filter was subjected to acid digestion using a CEM MARS 6 (microwave digestion system). Specifically, 50 mg of SPM was treated with 1.0 mL HF and 0.5 mL HNO₃ in pre-treated Teflon vessels, heated sequentially to 140°C and 185°C for 4 hours, and followed by dissolution in 2.5 mL of 40% HNO₃ at 135°C for another 4 hours. The digested solution was diluted to 50 mL with 2% HNO₃. The HM concentrations in the leachate of water samples and after acid mineralization were analyzed

using ICP-OES with an iCAP 7400 Duo spectrometer. Detailed information on the measurement method is available at EN ISO 11885 (International Standard Organization [ISO], 2019). The SPM characteristics of the particle size distribution with the determination of the median floc size (D50) were performed using a laser particle counter HIAC/ROYCO 8000A, according to the KND 211.1.4.039 recommendations (Ministry of Environmental Protection and Nuclear Safety of Ukraine [Mindovkillya], 1995).

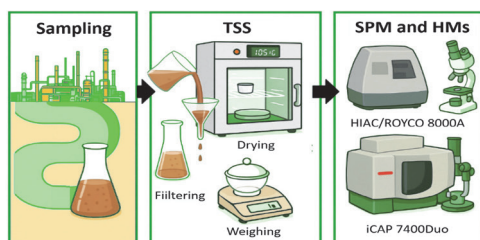


FIGURE 2. General scheme of sampling, sample preparation, and measurements

Source: compiled by the authors using AI graphical tools.

The distribution coefficient of heavy metals between the dissolved phase (water) and particulate (SPM) phases was calculated using the following formula (Cao et al., 2023):

$$K_d = \frac{C_{\text{SPM}}}{C_{\text{water}}},$$

where: K_d is a distribution coefficient [-], C_{SPM} is the concentration of HMs in the SPM phase [$\text{mg} \cdot \text{kg}^{-1}$], and C_{water} is the concentration of HMs in the dissolved phase [$\text{mg} \cdot \text{L}^{-1}$].

A higher K_d value (beyond 3) indicates a higher affinity of the metals for SPM or absorption, while a value below 3 means a higher affinity of the HMs for the dissolved phase.

The statistical processing of the research results involved determining the arithmetic mean (M) and the standard deviation (SD). In addition, the correlation between HM concentrations was determined using the Pearson correlation coefficient (r). The JASP software package Version 0.14.3 was used for the statistical processing.

Results and discussion

Variations in suspended particulate matter concentrations and characteristics

The distribution of TSS levels suggests a statistically significant increase in TSS levels during the warm period (Table 1). Thus, the $M \pm SD$ TSS concentration was $12.5 \pm 2.7 \text{ mg} \cdot \text{L}^{-1}$ in warm and cold periods, and the $M \pm SD$ TSS concentration was $18.3 \pm 3.0 \text{ mg} \cdot \text{L}^{-1}$. The observed seasonal variations can be attributed to multiple environmental and hydrodynamic factors (Walch et al., 2022). In the warm period, intensified biological activity, sediment resuspension due to enhanced flow turbulence, and increased precipitation-driven runoff contribute to elevated TSS levels. Conversely, during the cold period, reduced biological productivity and lower flow velocities may lead to greater sediment deposition, thereby reducing TSS concentrations.

TABLE 1. Concentrations of total suspended solids during cold and warm periods

Period	<i>M</i> [mg·L ⁻¹]	<i>SD</i> [mg·L ⁻¹]	min–max [mg·L ⁻¹]
Cold	16.2	7.3	5–30
Warm	21.7	7.1	10–35

Source: own work.

In the cold period, the distribution of SPM floc sizes was such that D50 equals 11.4 μm, indicating the predominance of small particles (Table 2). In contrast, during the warm period, D50 increases significantly to 24.8 μm, indicating the presence of larger particles and aggregation. Warmer conditions favor microbial activity, organic matter formation, and the strengthening of polymeric bonds, which contribute to the formation of larger flocs. In addition, the increased turbulence in the warm period promotes frequent particle collisions, leading to greater aggregation. Conversely, in colder conditions, reduced biological exudation and lower turbulence limit floc growth, resulting in the formation of smaller, more dispersed particles. Increases in TSS concentrations and floc size during the warmer months indicate a greater propensity for sediment transport and potential impacts on water clarity, nutrient content, and water resources.

TABLE 2. Morphological differences in the suspended particulate matter flocs in cold and warm periods

Feature	Cold period	Warm period
Visual appearance	small, dispersed, fine flocs	larger, more aggregated flocs
Dominant floc size range (min–max) [μm]	8–15	20–30
Floc size D50 [μm]	11.4	24.8
Structural compactness	more discrete, less aggregated	more compact and cohesive structures

Source: own work.

Pearson correlation analysis of heavy metal concentrations in the dissolved and the suspended particulate matter phases

The Pearson correlation analysis of HM concentrations in both the dissolved phase (water) and the SPM phase provides critical insights into the mechanisms governing metal transport, partitioning, and seasonal variability in aquatic systems. In the dissolved phase (Fig. 3), TSS exhibit weak or negligible correlations with most HMs, indicating that, in their dissolved state, these elements remain largely independent of the SPM. Therefore, solubility affects their distribution more than adsorption on SPM. However, strong positive correlations ($r > 0.7$) are observed among Fe, Mn, and Cr, suggesting a common geochemical origin or co-dissolution processes. Conversely, Cd shows weak correlations with most metals, which may indicate differences in its geochemical behavior, potentially driven by distinct sources, redox sensitivity, or competitive interactions with other ions.

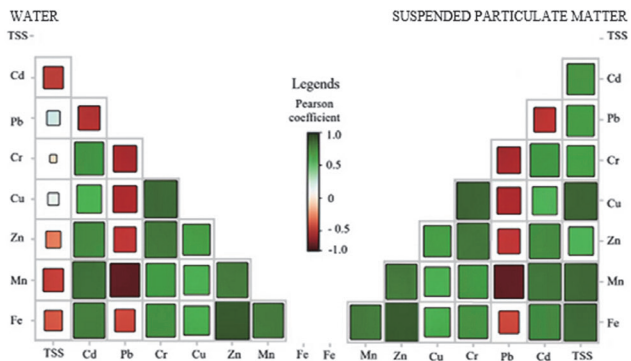


FIGURE 3. The Pearson correlation matrix of heavy metal concentrations in dissolved and particulate phases

Source: own work.

Negative correlations, such as those observed between Fe and Zn and Mn, further suggest that different metals follow distinct pathways in the dissolved phase, likely due to variations in their affinities for complexation with organic ligands or adsorption onto mineral surfaces. In contrast, in the SPM-associated phase (Fig. 3), HM correlation with TSS is substantially stronger, particularly for Fe, Mn, and Zn. This finding suggests that higher TSS concentrations enhance the adsorption of these metals onto suspended particulates, thereby increasing their transport in the particulate-bound phase. Strong correlations, particularly among Fe, Mn, Cr, and Pb ($r > 0.7$), indicate that these elements are primarily associated with iron and manganese oxides or organic-bound fractions within SPM. Meanwhile, Cd demonstrates only weak associations with other metals in SPM, further supporting the notion that it remains more mobile in its dissolved form compared to other HMs. During the warm period, increased TSS concentrations, as previously demonstrated (Table 1), enhance HM adsorption onto particulates, leading to stronger correlations between TSS and HMs in the SPM phase (Fig. 3).

Analysis of the distribution of heavy metals between dissolved and particulate phases

The seasonal distribution of HMs between dissolved and particulate phases, as detailed in Table 3, offers valuable insight into their environmental dynamics. The proportion of individual HMs within each phase and across periods demonstrates substantial variability, indicative of their distinct physicochemical properties, binding tendencies, and environmental reactivity under differing seasonal conditions. Fe and Mn consistently constitute the largest fraction of total metals in both the dissolved and particulate phases, a pattern that aligns with their prevalent occurrence as oxides and hydroxides in natural aquatic systems. However, distinct seasonal patterns emerge: during the cold period, elevated proportions of Cu, Zn, Pb, Cr, and Cd are found in the dissolved phase, whereas the warm period is marked by increased concentrations of Fe and Mn. Among the HMs analyzed (Table 4), the sequence of decreasing phase affinity follows: Fe > Mn > Cd > Pb > Cu > Zn > Cr.

TABLE 3. Content of heavy metals in dissolved and particulate phases during cold and warm periods

Phase/Period	HM content [%]						
	Fe	Mn	Cu	Zn	Pb	Cr	Cd
Water/cold period	47.4	25.6	4.7	3.1	6.9	7.8	4.5
Water/warm period	50.2	34.7	3.2	1.3	4.0	5.1	1.5
Suspended particulate matter/cold period	85.1	8.3	1.8	1.0	2.0	1.1	0.7
Suspended particulate matter/warm period	89.4	6.0	1.4	0.9	1.2	0.6	0.5

Source: own work.

TABLE 4. Distribution coefficient (K_d) values for heavy metals (HMs) and phase affinity interpretation

HM	K_d			Phase affinity
	min	M	max	
Cd	15.40	33.20	77.60	high absorption onto the solid phase
Pb	2.20	7.80	18.20	high absorption onto the solid phase
Cr	0.02	0.11	0.31	high dissolution into the liquid phase
Cu	0.30	1.21	2.05	intermediate absorption
Zn	0.12	0.63	0.71	tendency toward dissolution
Mn	53.10	104.30	502.10	high absorption onto the solid phase
Fe	203.10	455.60	707.20	very strong absorption onto the solid phase

Source: own work.

Intermediate K_d values observed for Cu and Cd suggest partial affinity toward SPM, although a substantial portion remains dissolved. This behavior is likely regulated by interactions with organic ligands and redox-sensitive solubility. Zn, exhibiting the lowest K_d values, demonstrates a clear preference for the aqueous phase, indicative of its higher solubility and weaker adsorption capacity (Antoniadis et al., 2007). These findings are consistent with previous studies indicating that Zn contamination is typically found in dissolved form rather than associated with particulates. In summary, the behavior of HMs in aquatic environments is largely governed by their chemical characteristics and environmental context (Liu et al., 2024). Fe and Mn serve as dominant sorbents and carriers within SPM, while HMs like Zn and Cr tend to remain dissolved due to lower sorption potential.

The observed seasonal fluctuations of the SPM and related HMs in the Styr River reflect the dynamics between hydrological, thermal, and geochemical processes. During the warmer months, elevated temperatures and increased biological activity promote the aggregation of finer particles into larger flocs (median size increasing from 11.4 μm to 24.8 μm), enhancing the capacity of SPM to bind with particle-reactive metals like Fe, Mn, Pb, and Cr (Fig. 4).

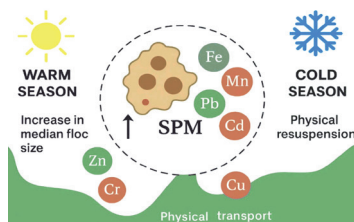


FIGURE 4. Complex pollution characteristics associated with suspended particulate matter and heavy metals

Source: compiled by the authors using AI graphical tools.

This trend aligns with findings by Liu et al. (2023), who reported that over 70% of HMs in the Yellow River were transported via SPM, with higher concentrations during rainy periods due to runoff-driven sediment loads. Our results also support the particulate-phase dominance of Fe and Mn ($K_d > 3$), while Zn and Cu remained primarily in the dissolved phase, suggesting differential geochemical behavior and binding affinities, consistent with reports by Xia et al. (2023) on the Yarlung Tsangpo River, where Cu and Zn exhibited weaker particulate associations.

Moreover, the stronger Pearson correlations between TSS and certain metals suggest that physical transport mechanisms (e.g., fluvial resuspension) rather than chemical transformations play a predominant role in metal mobility during high-flow periods. The K_d differentiation observed in our study corroborates the seasonal fluctuation models of HMs transport by Kuznietsov and Biedunkova (2023), emphasizing how SPM not only governs binding but also its seasonal flux across aquatic systems (Biedunkova & Kuznietsov, 2025). Neglecting SPM-associated fractions may significantly underestimate HM load, particularly during warm or flood-prone periods. Given the significant variation in HM behavior across physicochemical gradients and periods, long-term monitoring strategies should encompass both dissolved and particulate fractions for effective riverine metal pollution control.

Assessment of the distribution of heavy metals

The evaluation of HM distribution is essential for effective water quality management. Observations reveal significant temporal and spatial variations in HM concentrations, influenced by hydrometeorological events, fluvial discharge, and anthropogenic inputs (Fig. 5).

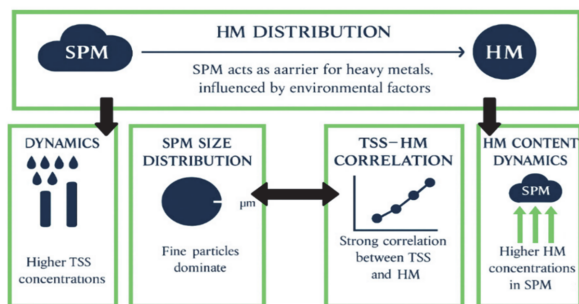


FIGURE 5. Assessment of the distribution of heavy metals (HM) in dissolved and particulate phases
Source: compiled by the authors using AI graphical tools.

According to Gong et al. (2016), TSS levels tend to rise during precipitation events, particularly when accompanied by moderate to high rainfall intensities and peaks, leading to increased TSS loads in runoff. In urban settings, dynamic TSS behavior is commonly observed in highway runoff, influenced by the proportion of fine (below 45 μm) and coarse particles, depending on prior dry periods (da Pereira Silva et al., 2024). Similarly, in tidal rivers, TSS exhibits high variability, which can be modeled to simulate sediment transport and estimate metal loads entering marine systems (Yao et al., 2021). Finer SPM particles possess a higher adsorption capacity for HMs, enhancing their role as carriers in aquatic environments (Li et al., 2023).

There is a well-established correlation between TSS and particulate-bound HMs such as Zn, Pb, and Cu in stormwater runoff. In river systems like the Yellow River, fine clay particles were found to significantly regulate HM concentrations, particularly during colder seasons. Comparable behavior was documented in the Xiaolangdi Reservoir, where resuspended sediments function as HM sinks, reducing downstream bioavailability (Dong et al., 2018). However, under certain geochemical conditions, SPM may also act as a source of HMs due to interactions with carbonate fractions or variations in organic matter.

Environmental factors such as salinity, sediment concentration, and dissolved organic matter significantly influence HM distribution between dissolved and particulate phases. In the Pearl River Estuary, HM concentrations were found to be lower upstream compared to downstream locations, particularly during warm periods, which was attributed to shifts in salinity and sedimentation rates (Du et al., 2019). TSS serve as an effective proxy for evaluating particle quality and size distributions in stormwater, assisting in the development of targeted mitigation strategies (Wang et al., 2018).

Conclusions

This study highlights the critical role of SPM for moving and separating HMs in water ecosystems. Seasonal variations significantly influenced both the concentration and morphology of SPM, with warmer periods promoting particle aggregation and enhancing the sorption capacity for particle-reactive HMs such as Fe, Mn, Pb, and Cd. In contrast, metals like Zn, Cu, and Cr remained predominantly in the dissolved phase, reflecting their lower affinity for SPM. The K_d and r analyses confirmed that metal mobility and phase distribution are strongly governed by environmental conditions, particularly temperature, turbulence, and biological activity. Ultimately, incorporating SPM-based assessments can improve the accuracy of ecological risk evaluations and inform effective strategies for surface water pollution management. The obtained parameters can be directly used for monitoring setups (including SPM and K_d in the list of indicators), determining TSS thresholds as sampling triggers, optimizing treatment plant and water treatment regimes, forming local standards and risk maps for the Styry sub-basin. The practical result will be an increase in the accuracy of environmental risk assessment, early warning of episodes of increased HM mobility, and support for management decisions in water use.

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Summary

A case study of complex pollution characteristics associated with suspended particulate matter and heavy metals. Suspended particulate matter (SPM) plays a pivotal role in the transport and separation of heavy metals (HMs) in river systems and has a significant impact on water quality and ecosystem integrity. This study examined the seasonal dynamics of total suspended solids (TSS), SPM particle size distribution, and the separation of HMs (Cd, Pb, Cr, Zn, Cu, Mn, Fe) between dissolved and particulate phases in the Styr River (Ukraine) throughout 2024. Analytical methods included gravimetric analysis, inductively coupled plasma optical emission spectrometry (ICP-OES), and laser-based particle sizing. The TSS concentrations and median floc size (D50) were substantially elevated during the warm season, enhancing sorption of metals onto SPM, especially for Fe, Cd, Pb, and Mn ($K_d > 3$). In contrast, Zn, Cu, and Cr exhibited stronger preferences for the dissolved phase. Pearson correlation analysis revealed strong associations between TSS and particulate-bound metals during warmer periods. Seasonal variations in hydrological and biological parameters significantly influence the physicochemical behavior of SPM and the separation of HMs. This is important for assessing the contamination of surface water with both dissolved and particulate phases of HMs.

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Investigation of failure modes and load-deflection of cold-formed steel connections using sub-assembly frame tests

Keywords: cold-formed steel, sub-assembly frame test, failure mode, load–deflection, experimental study

Introduction

Cold-formed steel (CFS) is shaped at ambient temperature through bending and rolling, unlike hot-rolled steel (HRS), which is formed at high temperatures. This process enables more precise shaping and sizing, making CFS ideal for construction applications where accuracy and consistency are essential (Dubina et al., 2012; Yu et al., 2019).

The benefits of cold-formed steel (CFS) in construction include its versatility, durability, and sustainability. CFS structures are lightweight yet remarkably strong, providing greater design flexibility and enabling the creation of complex architectural forms. Moreover, CFS is highly resistant to corrosion and requires minimal maintenance, making it a durable and eco-friendly building material. Its recyclability further contributes to sustainability, reducing waste and lowering the overall carbon footprint of construction projects (Johnston et al., 2018; Harini et al., 2020). Overall, the advantages of using cold-formed steel in construction are numerous, making it a popular choice among builders and architects.

However, there are some disadvantages; one major drawback is its higher initial cost compared to other building materials, such as wood or concrete. Additionally, cold-formed steel is more susceptible to buckling due to its thinness. Based on EN 1993-1-1 (European Committee for Standardization [CEN], 2005), CFS is categorized as a Class 4 section, due to a high width-to-thickness ratio.

A connection is part of a structure that is critical in ensuring stability and structural integrity. Proper connections can also help distribute loads evenly, prevent structural failure, and improve the overall performance of the construction project. Research on CFS connections has focused on apex and eaves (Pouladi et al., 2019; Chen et al., 2021; Wang et al., 2021; El-Hadary et al., 2022), trusses (Dar et al., 2020; Fitrah & Nofriyandi, 2020; Song et al., 2020; Bondok et al., 2021), and beam-column structures (Ye et al., 2019; Aminuddin et al., 2020; Firdaus et al., 2020a; Firdaus et al., 2020b; Seok et al., 2022). This recent study is limited to the beam-column connection.

Using finite element analysis, Seok et al. (2022) studied altering the through-plates to improve their usefulness in field applications. The numerical simulation results demonstrated that the proposed connection has an equivalent structural performance to the original design. For practicality, attaching a flange plate perpendicular to the diagonal edge is preferable to prevent buckling of the through-plate.

Research conducted by Ye et al. (2019) focused on an in-depth examination of the improvement of seismic behavior in beam-column bolted connections. Their research aimed to identify enhanced design solutions for CFS frames in connection areas susceptible to earthquakes through friction-slip mechanisms. Research demonstrates that the bolted friction-slip mechanism could significantly improve (by up to 200%) the flexibility, ability to dissipate energy, and damping coefficient of the connections, particularly for Class 3 and Class 4 CFS elements. The findings are used to identify optimal design configurations for enhancing the seismic responsiveness of the beam-column connections during intense

earthquakes. While Class 3 and Class 4 cross-sections do not meet the American Institute of Steel Construction specifications for intermediate and special moment frames, the proposed connection mechanism has demonstrated effectiveness in regions with high seismic activity.

This paper presents an experimental study using a sub-assembly frame test method to evaluate rectangular and haunched gusset plates. A top-seat angle was added to enhance connection resistance. This application used M12 bolts with a grade of 8.8, suitable for light steel profiles, as fasteners.

Material and methods

The flowchart of the current study, as shown in Figure 1, begins with preparation, where materials and equipment are organized. Next, assembling the specimens involves constructing the cold-formed steel beam-column connections according to the sub-assembly frame method. Measurement instruments, such as load cells and displacement transducers, are then set up to monitor specimen behavior. The study then examines failure modes to identify structural failure. Finally, load-deflection results are analyzed to assess the connection's performance under a monotonic load.

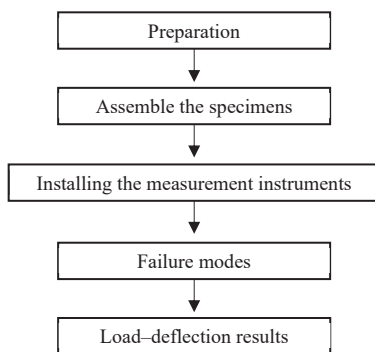


FIGURE 1. Flowchart of the current study

Source: own work.

The arrangement of CFS is back-to-back with double-lipped channel sections. The column section utilized C300, while C200 and C250 were used for the beam section. Each member was designed with grade 450 N·mm⁻², as specified in EN 1993-1-3 (CEN, 2006). The dimensions of the CFS sections are shown in Table 1 and Figure 2.

TABLE 1. Cross-section of cold-formed steel

Cross-section	Depth (d)	Thickness (t)	Flange width (b)	Lip (l)
	mm			
C20024	200	2.4	75	16
C25024	250	2.4	75	20
C30024	300	2.4	100	25

Source: own work.

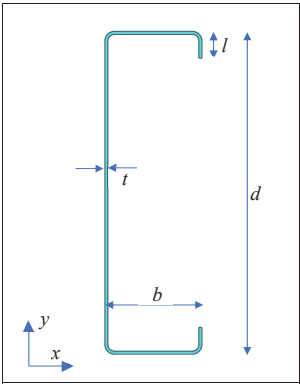


FIGURE 2. Cold-formed steel element

Source: own work.

Three distinct types of connections were employed: a rectangular gusset plate, a rectangular gusset plate with a flange cleat, and a haunched gusset plate. The gusset plate and flange cleat were fabricated from hot-rolled steel to connect the beam and column (Fig. 3, Table 2). Table 2 indicates that the abbreviation SAR denotes a sub-assembly frame test for a rectangular gusset plate; SARFC signifies a sub-assembly frame test with a rectangular gusset plate and flange cleat; and SAH represents a sub-assembly frame test for a haunched gusset plate. Presented here are a few essential terms, as follows: t_g denotes the thickness of the gusset plate, h_g indicates the height of the gusset plate, e_1 represents the horizontal distance of the bolt, e_2 symbolizes the vertical distance of the bolt, p_1 refers to the horizontal spacing of the bolt, p_2 relates to the vertical spacing of the bolt, and p_3 depicts the horizontal spacing of the bolt on both the column and beam.

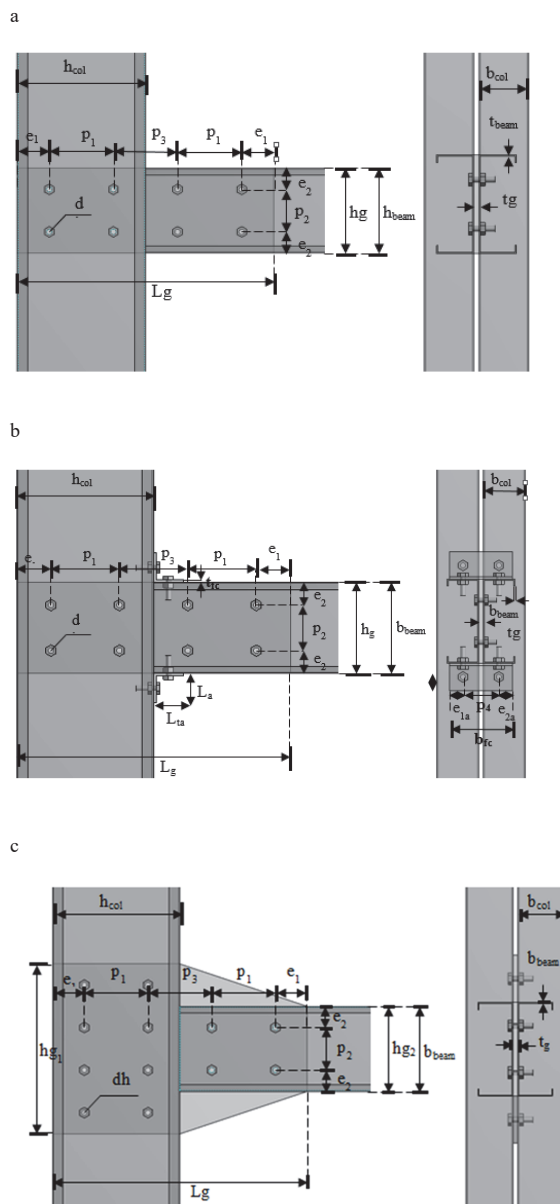


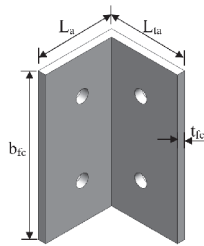
FIGURE 3. Configuration of beam-column connections: (a) rectangular gusset plate, (b) rectangular gusset plate with a top-seat angle, (c) haunched gusset plate

Source: own work.

TABLE 2. Specimen configuration details

Specimen	Beam	Column	Flange cleat	Connection details [mm]							
				t_g	h_g	L_g	e_1	e_2	p_1	p_2	p_3
SAR-200	C20024	C30024	–	10	200	600	75	50	150	100	150
SAR-250	C25024	C30024	–	10	250	600	75	50	150	150	150
SARFC-200	C20024	C30024	$L\ 65 \times 65 \times 160$	10	200	600	75	50	150	100	150
SARFC-250	C25024	C30024	$L\ 65 \times 65 \times 160$	10	250	600	75	50	150	150	150
SAH-200	C20024	C30024	–	10	400	600	75	50	150	100	150
SAH-250	C25024	C30024	–	10	550	600	75	50	150	150	150

Source: own work.



The dimensions of the flange cleat are shown in Figure 4 and Table 3. t_{fc} stands for the thickness of the flange cleat, and r_{fc} for the corner radius of the flange cleat. l_a indicates the length of the leg, l_{ta} represents the length of the top leg, e_{1a} refers to the horizontal edge spacing of the bolt, e_{2a} represents the vertical edge spacing, and b_{fc} shows the width of the flange cleat.

FIGURE 4. Flange cleat dimensions

Source: own work.

TABLE 3. Flange cleat connection details

Specimens	Beam	Column	Connection details [mm]							
			t_{fc}	r_{fc}	L_a	L_{ta}	e_{1a}	e_{2a}	p_2	b_{fc}
SARFC-200	C20024	C30024	6	5	65	65	32.5	32.5	85	160
SARFC-250	C25024		6	5	65	65	32.5	32.5	85	160

Source: own work.

The span in this study was 4,000 mm, a typical construction dimension. Six samples were tested, each containing two proposed specimens. The arrangement of the sub-assembly frame test specimen within the Magnus frame is shown, along with the precise locations of the load cell and spreader beam (Fig. 5). The Magnus frames were organized according to the specimen’s structural configuration. A pump-equipped hydraulic jack applied the load, with a load cell positioned at the center, and the spreader beam placed at two points separated

by a two-meter distance. The distance of the spreader beam is based on a quarter of the length of the beam span to ensure the bending condition of the beam. The load was incrementally applied to a value between 0.2 and 0.5 kN using a hydraulic jack.

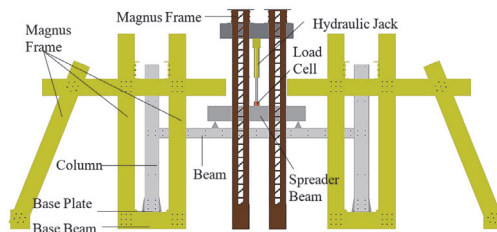


FIGURE 5. Schematic diagram of the sub-assembly frame test and boundary conditions

Source: own work.

The sub-assembly tests feature full-scale construction with a connecting beam spanning two columns positioned 4,000 mm apart. The gusset-plate connection is placed 1,500 mm from the column center, with the column height measuring 3,000 mm. The beam dimensions are categorized into two sizes: C200 and C250. The columns, on the other hand, have a standard section dimension of C300 with a grade of $450 \text{ N}\cdot\text{mm}^{-2}$, according to EN 1993-1-3 (CEN, 2006) specifications.

Eleven LVDT units are employed, with LVDTs 1–3 positioned at the beam's midpoint, precisely where the highest moment occurs. LVDTs 8–11 are mounted horizontally on both web columns, while LVDTs 4–7 are arranged to capture accurate vertical deformation (Fig. 6).

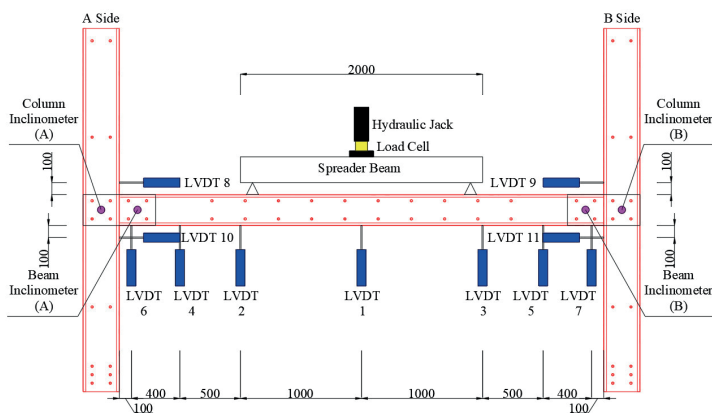


FIGURE 6. Instrument position on the specimen

Source: own work.

The test concluded upon observing either a visible failure mode or a sudden and significant displacement increase. The examination identified the failure mechanisms, including buckling at the column flange and substantial rotation of the connection due to bearing failure at the bolt hole. Additionally, thorough visual monitoring was carried out during all examinations, encompassing visual and physical inspections and video documentation for animation. Photographs were taken of each significant item, location, and incident.

Results and discussion

A monotonic load distributed at two points was applied to the beam (Fig. 7). The SAR-200 failure mechanism appears to occur in the compression zone of both the column flange and the bottom beam flange (Fig. 8). This is due to the bottom of the beam flange pushing toward the face of the column flange.



FIGURE 7. Full-scale test using the sub-assembly frame test method

Source: own work.

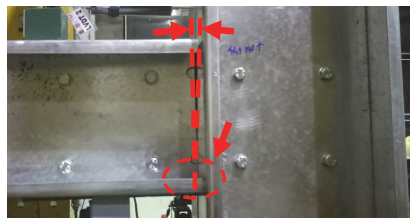


FIGURE 8. Deformation in the SAR-200 specimen

Source: own work.

The failure process of bending the web beam under the applied load occurred (Fig. 9). Lateral-torsional buckling of the beam caused crushing on the top flange and web. The midspan failure was due to twisting induced by this buckling.

The failure mode occurred because the top flange was the most vulnerable component (Fig. 10). The test ended due to lateral-torsional buckling, which led to a reduction in the applied load. The flange deformation occurred with compression observed at both the top and bottom due to the load applied to the beam (Fig. 11).

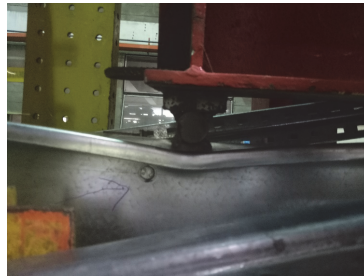


FIGURE 9. Failure on the beam's top flange

Source: Own work



FIGURE 10. Lateral-torsional buckling of the beam

Source: own work.



FIGURE 11. Crushing failure on the beam's top flange

Source: own work.

The bearing failure modes occurred on the joint for SAR specimens (Fig. 12). The failure occurred due to the thinness of the CFS section on both the beam and column.

For SARFC specimens, bearing failure also occurs on the beam and column. The bearing failure is shown in Figure 13. The bearing failure occurred due to the thinness of the CFS section.

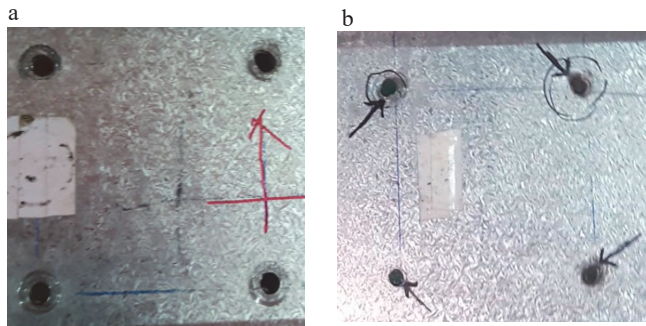


FIGURE 12. Typical failure modes of SAR specimens on (a) column and (b) beam
Source: own work.

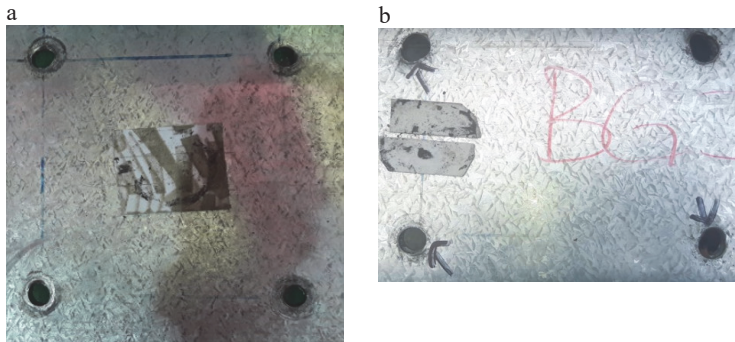


FIGURE 13. Typical bearing failure modes of SARFC specimens on (a) column and (b) beam
Source: own work.

Failure of the flange cleat was observed on SARFC specimens (Fig. 14). The bending of the column flange (Fig. 14) is due to the tensile force occurring on the bolts between the flange cleat and the column. In the compression zone, the buckling of the flange-cleat occurs. The failure indicated that the compression zone sustained greater damage to the flange cleat.

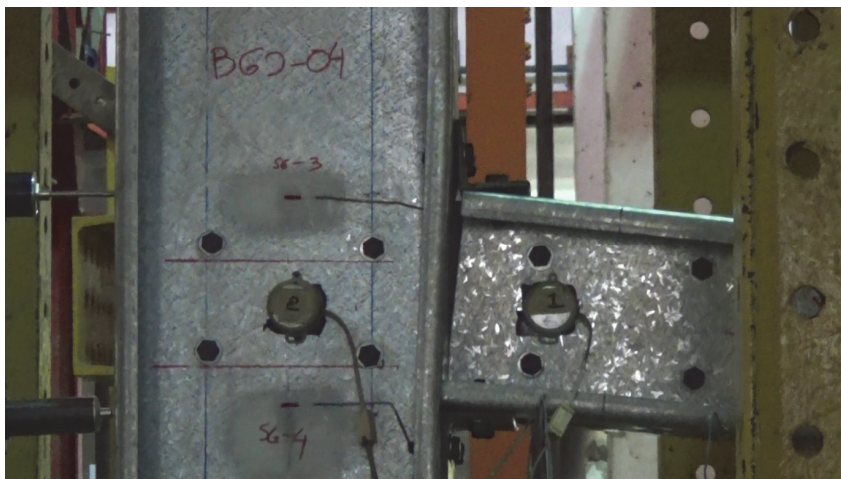


FIGURE 14. Typical failure modes of SARFC specimens on the joint
Source: own work.

The bearing failure mechanisms appeared on the joint for SAH specimens (Fig. 15). The thinness of the CFS section in both the beam and column led to insufficient load-bearing capacity, which ultimately resulted in the observed failures.

The experiment data were recorded in the data logger. The load was analyzed concerning deflection to establish the relationship between load and beam resistance. Graphs were generated to visualize this correlation between load and deflection.

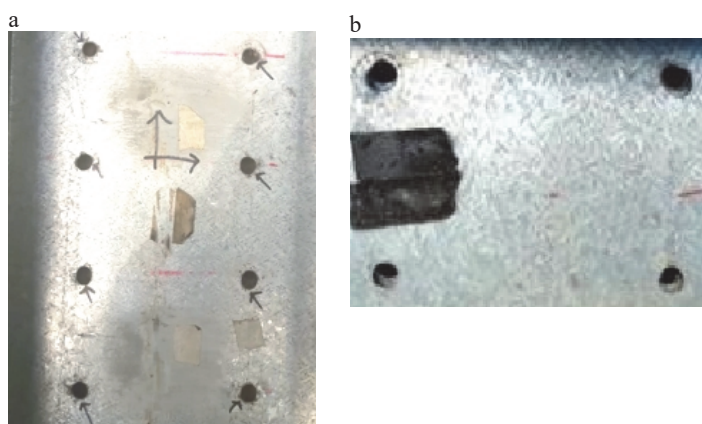


FIGURE 15. Typical failure modes of HG specimens on the joint
Source: own work.

The beam's deflection was measured with LVDTs positioned to determine the maximum deflection. Ensuring the load is at the deflection limit under the service limit state is crucial. Enhanced connection rigidity reduces beam bending. The load-deflection relationship was analyzed at the midpoint of the specimen, specifically at the position of LVDT-1. The maximum allowable deflection at the beam's midpoint is specified by Eurocode 3 (category: beams carrying plaster or other brittle finishing):

$$\delta_{360} = \frac{L}{360} = \frac{4,000+300}{360} = 11.94 \text{ mm} \quad (1)$$

$$\delta_{200} = \frac{L}{200} = \frac{4,000+300}{200} = 21.5 \text{ mm} \quad (2)$$

where: L is the distance between the midpoints of two columns. This limitation determines the maximum load that the beam can support with the designed connection.

The load for allowed deflection is obtained from the test using the acceptable deflection formula. P_{200} denotes the load corresponding to a deflection of $L/200$, whereas P_{360} indicates the load at a deflection of $L/360$. The graphs illustrate the correlation between load and deflection for the SAR-200 and SAR-250 models (Figs. 16 and 17). The SAR-200 maintains a maximum load of 68.82 kN and demonstrates a maximum deflection of 26.19 mm. The maximum allowable load for P_{200} is 59.04 kN, but for P_{360} it is 34.21 kN.

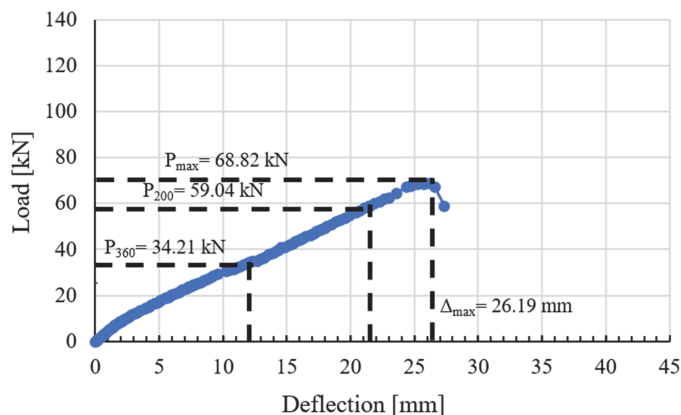


FIGURE 16. Load-deflection relation curve for SAR-200

Source: own work.

Figure 16 illustrates the load–deflection relationship for SAR-250. The maximum load, denoted as $P_{\max}$, is 83.58 kN, while the maximum deflection is recorded as 27.06 mm. The maximum allowable load for P_{200} is 70.79 kN, whereas for P_{360} it is 45.35 kN.

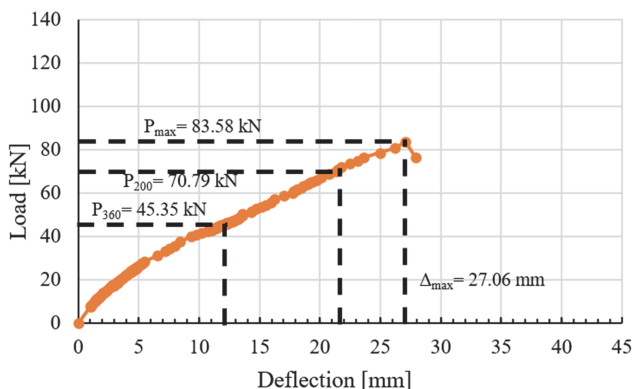


FIGURE 17. Load-deflection relation curve for SAR-250

Source: own work.

The SARFC-200 accommodates a maximum load of 103.63 kN with a deflection of 39.07 mm (Fig. 18). The maximum load for P_{200} is 64.30 kN, while P_{360} has a maximum load of 38.76 kN. The load-deflection graph of SARFC-250 indicates a $P_{\max}$ of 122.60 kN and a maximum deflection of 28.76 mm (Fig. 19). The allowable load limit for P_{200} is 98.03 kN, whereas for P_{360} it is 61.26 kN.

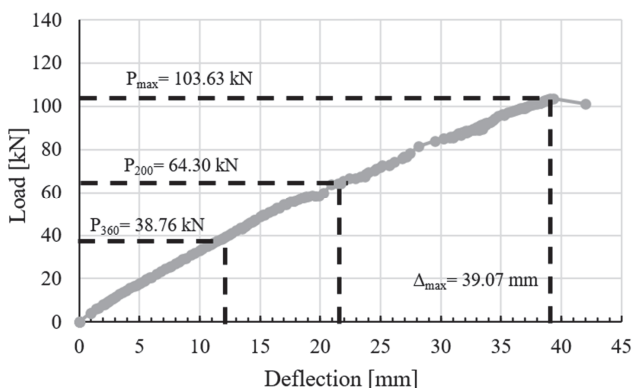


FIGURE 18. Load-deflection relation curve for SARFC-200

Source: own work.

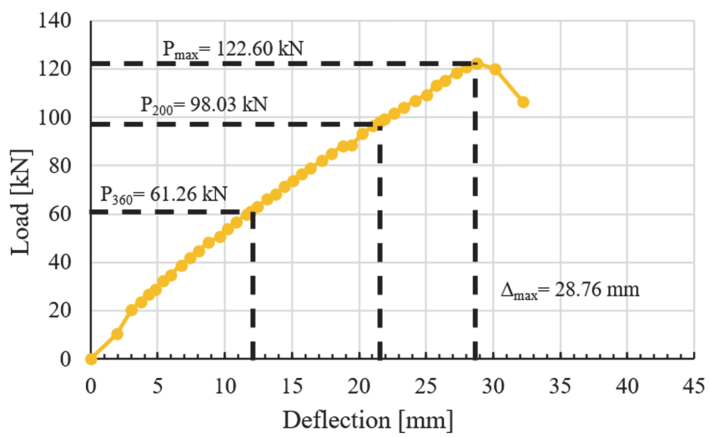


FIGURE 19. Load-deflection relation curve for SARFC-250
Source: own work.

The SAH-200 accommodates a maximum load of 108.86 kN with a deflection of 36.38 mm (Fig. 20). The maximum load for P_{200} is 74.34 kN, while P_{360} has a restriction of 45.40 kN. The load-deflection graph of SAH-250 exhibits a maximum load (P_{max}) of 124.73 kN and a maximum deflection of 25.78 mm (Fig. 21). The permissible load limit for P_{200} is 112.51 kN, whereas for P_{360} it is 68.64 kN.

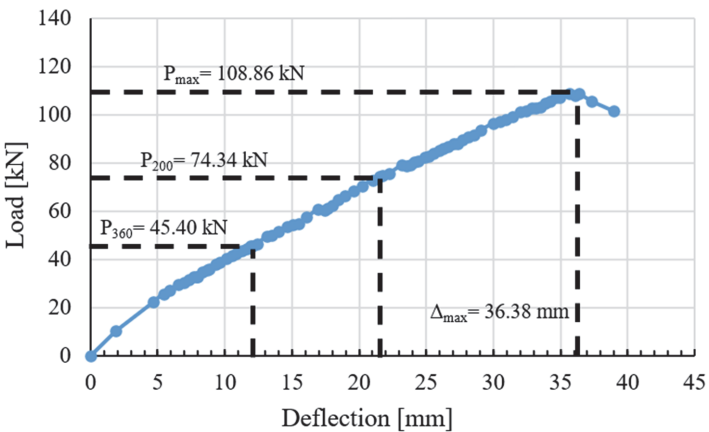


FIGURE 20. Load-deflection relation curve for SAH-200
Source: own work.

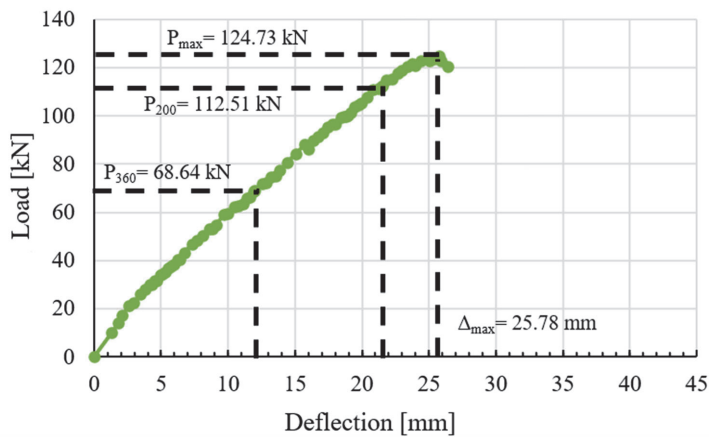


FIGURE 21. Load-deflection relation curve for SAH-250
Source: own work.

The SAH-250 specimen has the highest maximum load (P_{max}), while the SAR-200 specimen reveals the lowest (Table 4). SAR and SARFC connections possess a decreased load capacity relative to SAH, related to the gusset plate shape, which inhibits excessive beam deflection and results in more resilient connections. SAH-200 has a maximum load that is 55.03% more than that of SAR-200 and 6.94% higher than SARFC-200. SAH-250 has a maximum load that is 42.22% more than that of SAR-250 and 5.00% higher than SARFC-250. The SAH-250 has a maximum load that exceeds that of the SAH-200 by 14.58%. It shows that the height of the beam influences the beam deflection and the load capacity. The taller the beam, the more it increased the load capacity and decreased the beam deflection.

TABLE 4. Summary of load and deflection curves

Specimen	P_{max} [kN]	δ_{max} [mm]	P_{360} [kN]	P_{200} [kN]
SAR-200	70.22	26.72	34.29	59.43
SAR-250	87.70	28.39	46.25	71.50
SARFC-200	101.8	38.38	38.67	64.34
SARFC-250	118.8	27.87	60.73	97.14
SAH-200	108.86	36.38	45.40	74.34
SAH-250	124.73	25.78	68.64	112.51

Source: own work.

The utilization of flange cleats on SARFC also increases the load capacity significantly compared to SAR specimens. The SARFC-200 specimen has a load capacity 44.97% higher than the SAR-200, while the SARFC-250 has a load capacity 35.46% higher than the SAR-250. The influence of beam height shows that SARFC-250 has a load capacity 16.70% higher than SARFC-200.

Thus, the shape of the gusset plate, the height of the beam, and the utilization of flange cleats have a significant role in determining the load capacity and deflection of connections. Therefore, choosing the appropriate gusset plate shape, beam height, and the addition of a flange cleat is crucial for ensuring resilient connections in structural design.

Conclusions

A study was conducted on the SAFT of a cold-formed steel beam-column connection. The key conclusions are:

- SAH-250 has enhanced load capacity relative to other specimens, owing to the different type of gusset plate and an increased beam depth. SAH-250 has a maximum load that is 42.22% higher than SAR-250, while SAH-250 has a maximum load that exceeds SAH-200 by 14.58%.
- The utilization of flange cleats on rectangular gusset plates minimizes the difference of load capacity to haunched gusset plates by 6.94% for the C200 beam and 5% for the C250 beam. It shows that the utilization of flange cleats on SARFC also increases the load capacity significantly compared to SAR specimens by 44.97% for the C200 beam and 35.46% for the C250 beam.
- Failures observed in the specimens are consistent, including buckling of the beam's flange and web, as well as lateral torsional buckling.
- All of the specimens experienced bearing failure in the bolt hole in the beam and column. Additionally, a flange cleat failure occurred on the SARFC specimen.

Recommendations for future research include changing the beam profile to a sigma-type profile, or adding a stiffener on the web or flange to increase the stiffness of the CFS beam. Increasing the stiffness of the CFS beam may enhance its load-bearing capacity and overall structural performance. Future studies could also explore the effects of different materials or composite reinforcements to further improve the beam's resilience under various loading conditions. Additionally, investigating the impact of optimized geometric configurations and innovative

connection methods could yield significant insights into maximizing the efficiency of CFS beams. By combining these approaches, researchers may develop advanced solutions that not only improve structural integrity but also reduce material usage and construction costs.

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Summary

Investigation of failure modes and load-deflection of cold-formed steel connections using sub-assembly frame tests. Cold-formed steel (CFS) is a type of steel manufactured at ambient temperature using bending and rolling methods, enabling enhanced precision in shape and sizing. Its adaptability, resilience, and environmental sustainability render it very beneficial for building. Furthermore, its manufacturing method is more energy-efficient than conventional steel production and decreases emissions. Nonetheless, CFS has several limitations, including vulnerability to buckling owing to its slender profile. This research especially examines beam-column constructions with cold-formed steel connections. The paper discusses an experiment utilizing the sub-assembly frame test method to improve the connection's resistance by emphasizing haunched gusset plates, rectangular gusset plates, and a combination of rectangular gusset plates with the top-seat

angle. The results demonstrated that the lateral-torsional buckling occurred on the beam. The connection's bolt hole exhibited a bearing failure, and the flange cleat also experienced buckling. Graphs were employed to examine the load and deflection data, with maximum load and deflection values documented for each specimen. The result shows that the type of gusset plate, the height of the beam, and the utilization of flange cleats have an impact on the performance of the connection.

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