

REPAIR OF CRACKS IN CONCRETE WITH THE MICROBIAL-INDUCED CALCITE PRECIPITATION (MICP) METHOD

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Abstract

In this study, the microbiologically-induced calcium carbonate precipitation (MICP) method was employed to examine its potential for repairing cracks in concrete. In addition, specific gravity and porosity values were measured to examine the effect of calcite formations on concrete surfaces and microstructures. Bacteria-supplemented concrete repaired cracks up to 0.4 mm wide by filling them with CaCO₃. Furthermore, this study not only examined the healing of the width but also the length of cracks. However, as the width of the treated cracks decreased, their length increased. This indicated that the MICP treatment is more effective in a limited crack range. Specific gravity values increased, and porosity values decreased in concrete supplemented with calcifying bacteria. SEM analyses showed that calcite is a bacterial product that forms a very tight bond with a cement gel and that calcite fills visible cracks and voids and creates more of a void-free and undamaged concrete structure.

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Key words

- Crack repair,
- Self-healing,
- Bacterial concrete,
- MICP,
- Porosity.

1 INTRODUCTION

Concrete is the construction material most preferred due to its low cost, desirable visuality, ease of production, high durability, and compressive strength. It is exposed to various external factors and forces throughout its service life. These forces often cause cracks, spills, and separations on concrete surfaces. Cracks formed on a concrete surface are classified as structural cracks (due to design errors, its application, and inspection problems) and non-structural cracks (quality of the material used, freezing/thawing, ambient conditions, reactions, chemical effects, additional loads, and other environmental factors). Although the compressive strength values desired are achieved in structural concrete today, cracks still occur due to the reasons listed above and damage the durability properties of the concrete. Therefore, treating cracks encountered on a concrete surface appropriately is a necessity

because cracks formed as a result of this damage are known to negatively affect the mechanical properties and durability of concrete (Yıldırım and Özhan, 2023). Although cement offers autogenous improvements in cement-based composites, such improvements are limited in width (Danner et al., 2019; Wang et al., 2014b) and mechanisms proposed. There was no effect of varying silica fume (4%, 12%). Materials used to repair concrete cracks are usually polymer or cement-based products but repairs with these materials are costly. Such repairs result in extra costs to the structure over the long run (Muhammad et al., 2016; Van Breugel, 2007). In addition, cement and similar materials used for crack repairs and long service life cause high CO₂ emissions. However, the concrete's composition contains natural components, not CO₂-generating emissions, except for cement. Researchers have thus started to use the microbial calcification method (MICP) as an autonomous healing strategy, as it is a more natural, environmentally-frien-

dly, and economical method (Bachmeier et al., 2002; Choi et al., 2017; De Muynck et al., 2008; Jongvivatsakul et al., 2019; Van Tittelboom et al., 2010; Wu et al., 2019).

Carbonate mineralization is a frequently observed mechanism in nature. For example, the growth of corals, seashells, the protective shells of some animals, and bones are formed or regenerated as a result of bio-mineralization. The formation of bacterial calcium carbonate depends on the metabolic activity of bacteria (Hammes et al., 2003). Feeding the bacteria to increase its metabolic rate also increases carbonate mineralization. Many bacteria in nature can hydrolyze urea to form calcification. The production of calcium carbonate (CaCO_3) by bacteria can be considered as an ideal process for MICP. The bacterial urease enzyme initiates carbonation (Equation 1). In the presence of calcium ions and suitable environmental conditions, calcium carbonate is formed (Equation 2). The suitable conditions may depend on many variables, such as the type of bacteria, ambient temperature, pH value, oxygen and calcium concentration in the environment (Okwadha et al., 2010). Calcium carbonate formed as a result of this reaction is known to have a high degree of purity (Beveridge, 1988).



The biological calcification created by bacteria is shown schematically in **Fig. 1**.

Filling the voids and cracks of concrete with calcium carbonate produced by bacteria is remarkable. Cracks filled with CaCO_3 formed as a result of Equations 1 and 2 are also important for reinforced concrete structures, because 80% of all structures in the world are reinforced concrete structures and must therefore be protected against corrosion through cracks

(Joshi et al., 2017). In concrete repair studies using the MICP method, the bacillus genus of bacteria is particularly preferred, because this genus can preserve itself for up to 200 years unless there is an undesirable off-limit condition (Wang et al., 2014a). Considering the long service life of building materials, it is important that bacteria can survive for a long time when used in a material. Any bacteria that have the ability to produce calcite, a long-living alkali-resistant substance, can be used as an additive in concrete. *Bacillus* species with spore stages in their life cycle are perfect candidates for such applications of MICP.

Various methods are used for bacteria supplementation in concrete mixtures. These methods include spraying, injection, and its direct addition to a mixture. The most effective and practical method is its direct addition to a cement mixture because both external and internal cracks can be treated without the need for any additional treatment (Le Metayer-Levrel et al., 1999). Therefore, in the present study, *Bacillus megaterium* was directly supplemented in concrete mixtures. This method was successfully used in crack repair applications, and the concrete cracks were healed by filling them with CaCO_3 (Chintalapudi et al., 2017; Feng et al., 2021; Van Tittelboom et al., 2010; Wiktor and Jonkers, 2011; Zhang et al., 2017). In bacterial concrete, self-healing behavior positively altered other properties such as compressive strength, water absorption, and permeability, as well as crack repair (Andalib et al., 2016; Krishnapriya et al., 2015).

However, MICP is still a developing treatment technique. This study examined bacterial self-healing not only in the width but also in the length of cracks. Therefore, the effective width and length of the cracks were also examined. In addition, the effectiveness of the MICP treatment, depending on the curing time, was investigated by examining the cracks on the bacterial concrete samples that were cured for different amounts of days. In addition, the effect of calcite formed by *Bacillus megaterium*, one of the bacteria with the highest urease capacity, on the specific gravity and porosity level of concrete was investigated. The image of this bacterial product

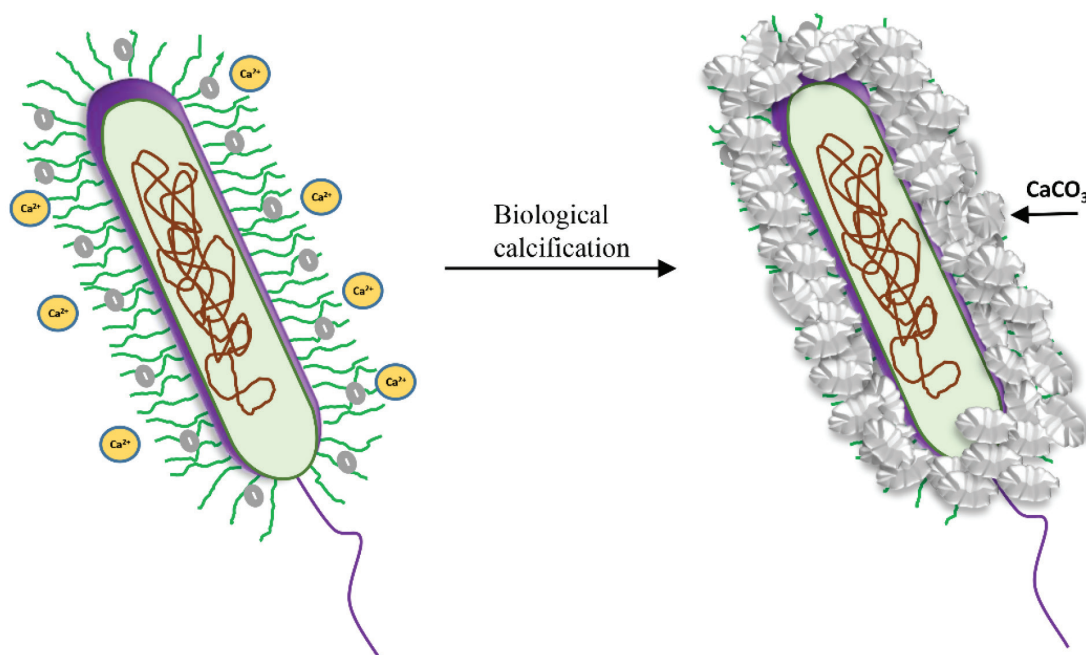


Fig. 1 Schematic representation of microbially-induced calcite precipitation by bacteria.

calcium carbonate, was examined under a scanning electron microscope (SEM), and its compatibility with the concrete matrix was observed.

2 MATERIALS AND PREPARATION OF SAMPLES

2.1 Microorganisms, growth conditions and media

Bacillus megaterium, which was selected as the bacterium identified to have a high degree of urease activity, was obtained from the Culture Collection of Refik Saydam Hygiene Institute (Ankara, Turkey). The cells were maintained at -80°C in 50% (v/v) glycerol and cultured on agar nutrient plates at 30°C for 24 h. Pure colonies were transferred into sterile nutrient broth media, and the culture was incubated at 30°C with agitation at 150 rpm for 24 h. To carry out the experiments, a nutrient broth-urea (NBU) medium (8 g nutrient broth, 2% urea solution and 25 mM CaCl₂) was used as described by (Achal et al., 2009), and the final pH of the medium was adjusted to 8.0. The nutrient broth was autoclaved at 121 °C for 15 minutes and supplemented with a filter-sterilized 2% urea solution and 25 mM CaCl₂.

2.2 Concrete materials and proportion of mix

CEM I 42.5R Portland type cement was used in all the mixtures. The aggregates were prepared and used in accordance with TS 706 EN 12620 standards (TS 706 EN 12620+A1, 2009). The specific gravity of the coarse aggregates used was 2.75, and the specific gravity of the fine aggregates was 2.65. Tap water was used in all the mixtures. The water/cement ratio in the concrete mixture is 0.45. The weights of the materials used in the mixtures for 1 m³ are given in Tab 1.

Tab. 1 Composition of the concrete mixture

Materials	Volume(kg/m ³)
Cement	195
Water	433
8-16mm aggregate	618
4-8mm aggregate	353
0-4mm aggregate	766

2.3 Preparation of concrete sample

The production of the concrete was performed in accordance with TS 802 standards (TS 802, 2016). Two types of concrete were produced in this study, i.e., bacterial concrete (BC) and the control concrete (CC). In the bacterial concrete, the bacterial cells with a density of 3×10⁶ cells/mL were inoculated into the NBU medium while making curing water for the concrete samples. No bacteria were added to the curing water in the control concrete. Except for the bacteria inoculation, the production conditions and materials were exactly the same. Fresh mixtures were poured into molds (100 x 100 x 100 mm)

and removed from the molds after 24 hours. The samples were kept in the curing liquid (the NBU medium) for the days of the experiment (28 and 90 days). Three replicates from each type of concrete were used for each experiment.

3 TEST AND METHODS

3.1 Generation and healing of cracks

It was expected that the cracks would be healed as long as the required nutrients and oxygen were taken by the bacteria present on the surfaces of the crack (Seifan et al., 2017). The 7-day concrete samples cured in the curing liquid (NBU medium) were cracked using a compressive strength press. The cracks were generated on three replicates from each series. The range of the width of the cracks generated on the concrete was between 0.1 - 0.5 mm (Fig. 2). For this crack formation, the compressive strength value of the concrete was determined first. Then, 90% of this compressive stress was loaded on the concrete samples, and cracks were formed. The cracked concrete was placed back into the NBU medium and cured for 28 days and 90 days. At the end of 28 and 90 days, the changes in the crack structures were recorded by photographing them. At the same time, CC and BC samples with out any cracks were also kept in the same NBU medium as controls. The medium liquid was renewed every 28 days.

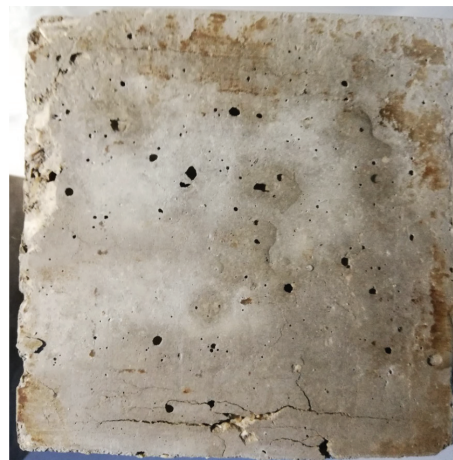


Fig. 2 The cracks generated on the concrete surface.

3.2 Specific gravity

Since the products formed in the pores and cracks on the surface of the concrete, it was expected to form a filled structure; specific gravity tests were applied to determine the rate of the filling. According to Equation 3, the weights of the concrete samples in the saturated water weights were measured, and the specific gravity of each specimen was determined.

$$\delta = \frac{(W_{sat})}{(W_{sat} - W_{wat})} \quad (3)$$

where: δ is the specific gravity; W_{wat} is the weight in water of the saturated sample; and W_{sat} is the weight in air of the saturated sample.

3.3 Porosity

The porosity was measured to determine the effect of bacterial products formed in the concrete samples cured for 28 days. After 28 days of curing, the saturated in water and oven-dry weights of the CC and BC concrete samples were determined. The porosity values were obtained with the use of Equation 4.

$$P = \frac{(W_{sat} - W_{dry})}{(W_{sat} - W_{wat})} \quad (4)$$

where: P is the apparent porosity; (%) W_{wat} is the weight of the saturated concrete sample in water; W_{sat} is the weight of the saturated concrete sample; and W_{dry} is the weight of the oven-dried concrete sample.

3.4 Scanning electron microscopy (SEM)

The SEM analysis was conducted with the use of a Zeiss / Gemini 300 Scanning Electron Microscope (BTU, Turkey). Fragments taken from the bacteria-supplemented concrete samples were examined. The widths of these structures were between 5-15 μ m. The samples were dehumidified and vacuumed before the SEM analysis was performed.

4 RESULTS AND DISCUSSION

4.1 Formation of CaCO_3 and Healing of Cracks

To monitor the calcium carbonate production of the bacteria, the bacteria-supplemented concrete samples cracked by applying stress were cured for 28 days and 90 days. The control concrete (CC) and bacterial concrete (BC) samples were

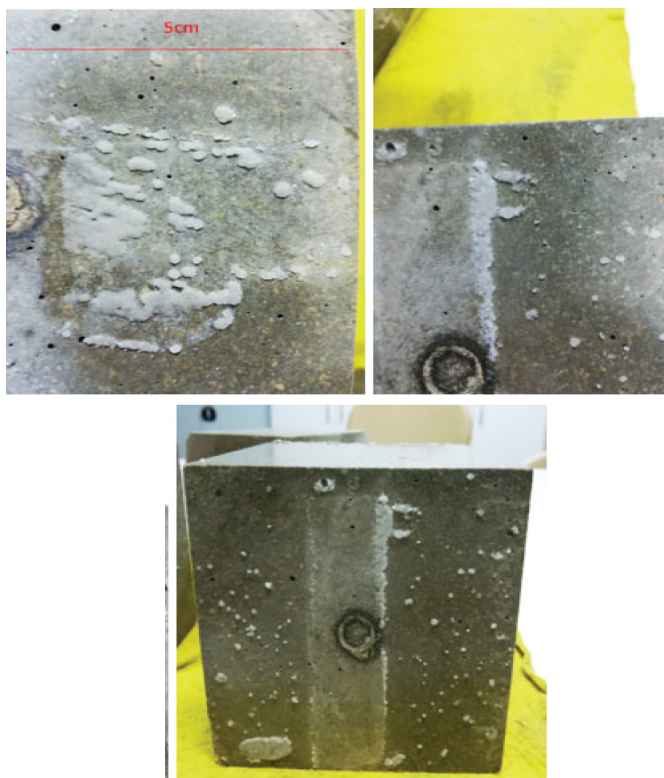


Fig. 3 CaCO_3 products formed on the surface of the 28-day BC concrete samples.

also treated the same with the bacteria-supplemented concrete samples. CaCO_3 formations were not observed in the CC samples. However, CaCO_3 products were formed in the crack-free BC concrete samples (Fig. 3). We found that the bacteria added to the concrete mixtures led to calcification after curing. We also observed that the CaCO_3 on the concrete samples tended to form calcification in the pores of the concrete samples. At the same time, the calcification products were firmly attached to the concrete samples and were difficult to scrape off.

Despite the decreased rates of calcification during the incubation, the bacterial calcification formations continued to occur after 28 days. When the surface of the 7-day, 90-day uncracked, and 100 mm-wide BC samples were compared, the surface of the sample in the 90-day uncracked sample became filled due to the calcification of the bacteria (Fig. 4). The 90-day bacterial activity was apparent as more whitish areas were on the sample compared to the 7-day samples.

After cracks were formed on the BC and CC concrete samples by applying compressive stress on the seventh day of the curing, the concrete samples were further cured for 28 and 90 days. During the curing process, the crack healing process was observed by photographing it. It is well known that even

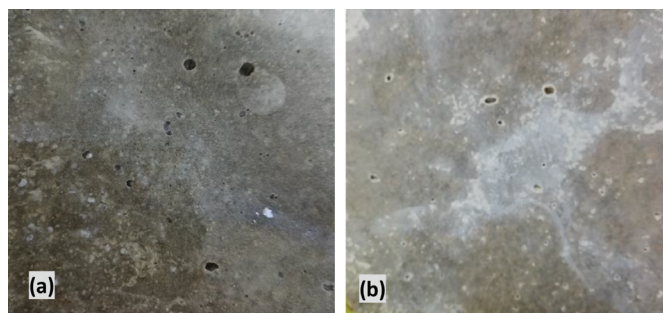


Fig. 4 Formations on the surface of the 7-day (a) and 90-day (b) BC concrete samples.

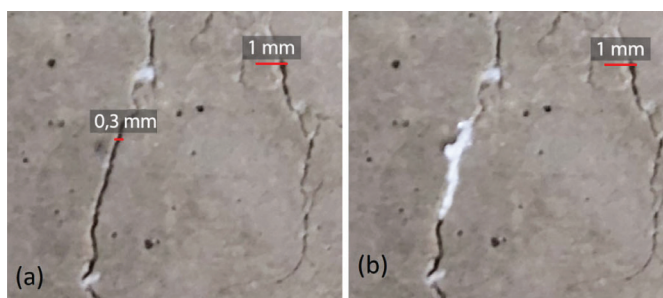


Fig. 5 Healing of crack (0.3mm) after 28 days of curing: (a) initial (b) final.

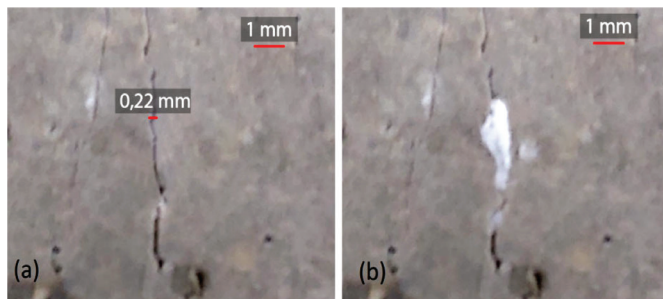


Fig. 6 Healing of crack (0.22mm) after 28 days of curing: (a) initial (b) final.

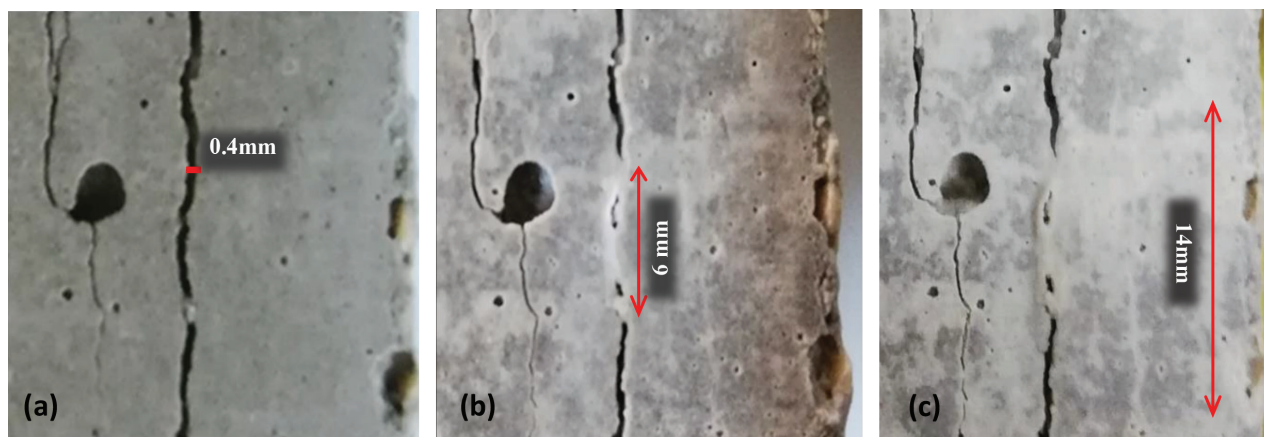


Fig. 7 Initial (a), 28-day (b), and 90-day appearance (c) of a 0.4 mm wide crack on the surface of BC.

cracks that do not pose a risk in terms of structural safety can cause durability problems over the longterm. Therefore, the healing of the various sizes of the crack widths is important. We observed that 28 days were enough to heal cracks of 0.3 mm (Fig. 5) and 0.22 mm (Fig. 6).

The initial, 28-day, and 90-day states of the cracks were examined in all the concrete samples to determine the long-term changes in the crack healing processes. The widest crack width repaired in this study was 0.4 mm and is shown in Fig. 7. Significant improvements in the healing of the crack were observed as early as 28 days (Fig. 7b), and a more general improvement was detected in the 90-day cured concrete samples (Fig. 7c). Acceptable improvements were particularly observed within the 14 mm-long region on the surface of the edge of the concrete sample. The voids and cracks in those structures were filled with calcite, and a white surface was formed. It was also observed that the color of the concrete sample turned from gray to whitish, indicating heavy calcification. Also, Qian et al. (2021) repaired cracks with a width close to the width of the cracks healed in our study using MICP.

Fig. 8 presents another concrete sample whose long-term behavior was examined. On this other concrete sample there were two 7 mm (0.20 mm width) and 15 mm (0.12 mm width) long cracks (Fig. 8). After 28 days, these two cracks had healed. In Fig. 8 (c), it was seen that the microstructures and micro cracks on the concrete sample were mostly filled with calcite. Similarly, to other cured concrete samples, the surface color turned white. In addition, it was also found that as the width of the

crack increased, the length of the healed crack decreased. The longest healed crack was observed to be 0.12 mm wide, where the amount of the width was most restricted. Thus, the MICP treatment was found to be more effective in more limited crack ranges.

In previous studies (Yildirim et al., 2023; Qian et al., 2021), it had been observed that similar widths of cracks had been repaired with the MICP method. Slight differences were mainly attributed to different species of bacteria and to the conditions present in healing the cracks. Chintalapudi et al. (2017) used *Bacillus pasteurii* as a biological agent in their study and observed healing in cracks in 56 days. Van Tittelboom et al. (2010) used *Bacillus sphaericus* in their study and also used different media for the bacterial activity. These factors had an impact on the closure of the cracks. Zhang et al. (2017) reported that bacterial cultures and conditions were effective in a crack closure. In the study of Wiktor et al. (2011), healing of cracks at the end of 100 days were observed. Feng et al. (2021) achieved the final healing of cracks after 28 days of the application of the MICP. In this study, although the crack closure was higher in the first 28 days, it continued for 90 days, but the type of bacteria and medium used must be considered.

The cracked and uncracked concrete samples were compared in terms of the CaCO_3 products, and it was observed that the CaCO_3 formations were primarily encountered in cracks. Such formations play a great role in the effectiveness of crack treatment. H_2O (water) and O_2 (oxygen) could pass more easily through the cracks by the capillary route, thus creating better conditions for the bacteria there.

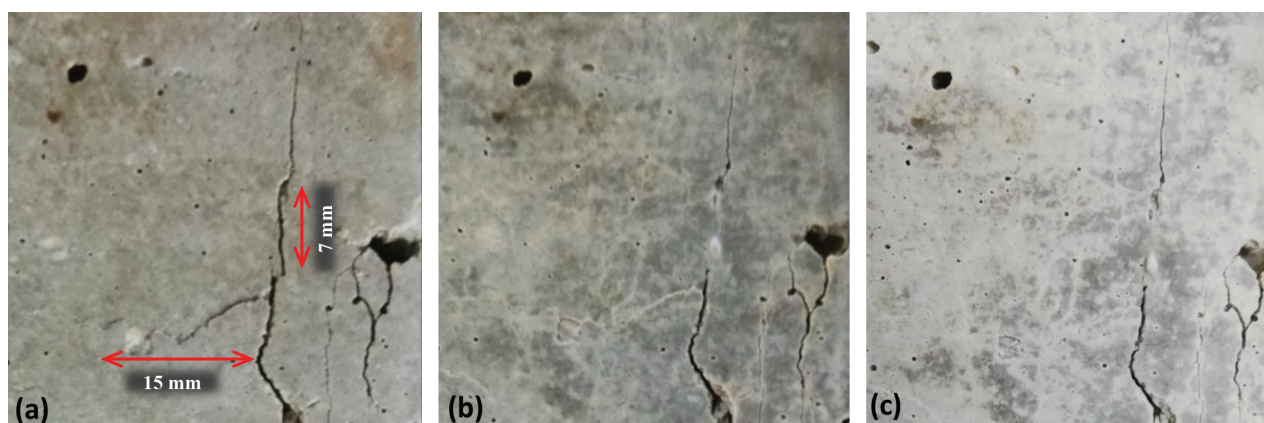


Fig. 8 Initial (a), 28-day (b), and 90-day (c) appearance of crack generated on the BC specimens.

Owing to these better conditions, the bacteria in the cracks use their microbial enzymes to create CaCO_3 (Jonkers and Thijssen, 2010; Ujike et al. 2010). Likewise, in our study, while the calcite formed in the uncracked concrete samples was dispersed in the voids and on the surface, it was formed primarily on the cracks in the concrete samples. In previous studies, it was reported that these calcites formed on concrete surfaces and that the filled-in cracks improved the mechanical properties and durability of the concrete. (Reddy and Revathi, 2019; Yildirim et al., 2023; Özhan et al., 2020).

4.2 Specific gravity

Calcium carbonate (CaCO_3) produced by the MICP method occurs in micro cracks and on surfaces, thereby creating an extra filling in a concrete sample. Therefore, the specific gravity values of the cured concrete samples increases. The specific gravity values of the concrete samples were calculated with the use of Equation 3 and the weights provided in **Table 2**. The specific gravity was calculated as 2.32 for the BC concrete samples and 2.30 for the CC concrete samples. The bacteria-induced CaCO_3 formations yielded this change in the specific gravity. Thus, it was observed that the bacteria generated a heavier and fuller volume. Fuller concrete volumes are known to have higher strengths and are more impervious to water. In this respect, it is important that calcite be formed in an amount that will increase the specific gravity of the concrete.

Tab. 2 Weights of the control and bacterial concrete

Sample	W _{saturated} (g)	W _{water} (g)	W _{dry} (g)
CC	2411	1364	2299
BC	2435	1386	2336

4.3 Porosity

A more void-free structure is possible by filling the pores with the CaCO_3 produced by the bacteria. The values given in Table 2 were used in Equation 4 to estimate the porosity values. The average porosity value was estimated at 10.72% for the CC samples and 9.43% for BC concrete samples. As a result, the calcifying bacteria also had an effect on the surface porosity of the concrete samples. The more void-free structures on the concrete samples after curing were also detected on the porosity levels. Our results clearly show that the CaCO_3 formed reduced the porosity values and filled the structure. In line with this study, the studies of Siddique et al. (2016) and Rao et al. (2017) showed that calcite produced by bacteria reduces porosity values. Porosity is crucial for the service life of concrete, as it is a property of concrete that primarily affects its water absorption properties.

4.4 Scanning electron microscopy (SEM)

The CaCO_3 formations created by the MICP method were examined under SEM. The crystallized structure of the calcium carbonate is clearly shown at 50X and 200X magnifications in Figs. 9 and 10. It is a stable product thanks to its crystalline structure.



Fig. 9 Microstructure of the calcium carbonate precipitation (CaCO_3) on the BC samples at 200X magnification.

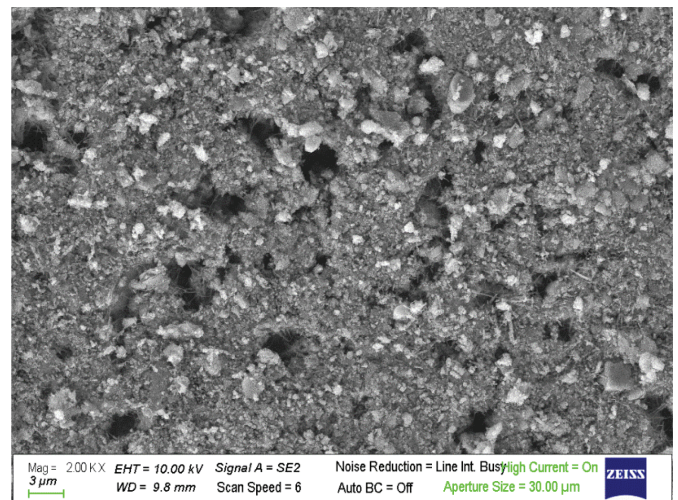


Fig. 10 Precipitated CaCO_3 on the surface of the concrete samples

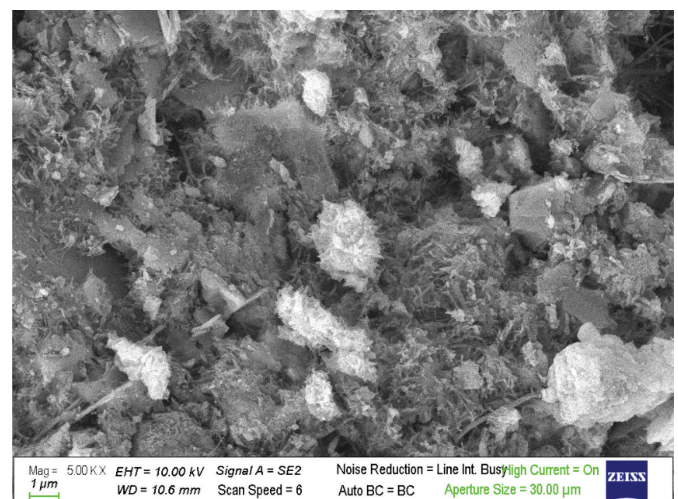


Fig. 11 Microstructure of bacterial concrete (BC)

When examined with 5.00KX magnification, the CaCO_3 in Fig. 11 revealed itself with its white color. In addition, the structure created with the CSH gels in the photograph and how

it filled the voids at the micro level was remarkable. In addition to filling pores and cracks, a strong bond was observed to have been established in the concrete structure, thanks to its compatibility with the CSH gel.

5 CONCLUSIONS

The addition of bacteria to a concrete mixture as a biological additive has positive effects on concrete structures, i.e., a more filled-in surface and the healing of cracks. The healing of the cracks was observed just a week after the curing process. After 28 days, the various cracks between 0.1 mm and 0.4 mm were completely healed. However, it was determined that the bacteria provided a longer closure at the narrower crack widths. This indicated that the MICP treatment is more effective in a limited crack range. In addition, calcite was formed in the bacterial samples that did not contain cracks, leading to more filled concrete surfaces. It was observed in the precracked concrete samples, that calcite tended to form

in cracks, so it was concluded that crack repair could be performed effectively in bacteria-supplemented concrete. Cracks in narrow areas present more surface area for bacterial activity. Moreover, calcite formations were also encountered on the surface and internal structure of the concrete, and such formations increased the specific gravity values and decreased the porosity values of the concrete samples. The microstructure of the calcium carbonate produced by bacteria in the bacterial concrete was investigated under SEM. Depending on the calcium carbonate mineralization, the formations and changes in the microstructure of the concrete were clearly observed. Further research is recommended with the use of different bacterial species and examining the effect of fibers to restrict crack widths.

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BEHAVIOR OF AN EMBANKMENT ON STONE COLUMN-REINFORCED SOFT SOIL

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Abstract

Reinforcing soft soils with good drainage and load-bearing material is a common practice in the construction of embankments. Stone columns are among the granular materials that accelerate consolidation settlements due to their high drainage capacities. This study aims to investigate the behavior of an embankment on stone column-reinforced soft soil. The embankment was analyzed using the Plaxis 2D Finite Element Method. The columns were converted into equivalent walls using the plane strain conversion method. This method was validated with field measurement results from previous studies and showed good agreement. According to the research findings, stone columns significantly reduced deformations and improved consolidation. For instance, the ultimate settlement was decreased from 54.3 mm to 33.1 mm for 10% and 20% area replacement ratios, respectively. As the safety analysis verified, basal geosynthetics contributed substantially to the embankment's stability. Furthermore, the safety factor values increased for all stages of the construction, and the global safety factor increased at the end of the construction. Stone columns and basal geogrid-reinforced soft soil can reduce settlement and increase the stability of an embankment.

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Key words

- Embankment,
- Plaxis 2D Finite Element Method,
- Safety Factor,
- Soft Soil,
- Stone Columns.

1 INTRODUCTION

Geotechnical structures founded on soft soil require significant attention due to their high water content, low shear strength, low permeability, and unique structural features. Excessive compressibility is a characteristic of soils with undrained shear strength below 15 kN/m². Moreover, applying too much structural weight to weak ground can generate excess pore pressure and lead to significant deformations. Buildings constructed on soft soils are prone to excessive deformation and slope failure, particularly deep-seated failure (Deshpande et al., 2021). As land values continue to rise and suitable sites for construction become scarce, engineers look

in situ improvement of weak soil deposits. Various ground improvement techniques have been employed to use marginal areas to construct buildings and other infrastructure. Among these techniques, stone columns have proven to be effective and reliable (Dheerendra et al., 2013).

Stone columns are commonly employed to enhance the load-bearing capacity of weak soil and minimize the subsidence of buildings constructed on them. Their analysis and design are typically based on simplified empirical rules, which consider the results of model experiments, on-site tests performed on individual stone columns, and previous experience (Lee and Pande, 1998). Stone columns are extensively used to improve soft clay deposits by increasing bearing capacity,

accelerating consolidation, and controlling settlements (Gu et al., 2022; Zheng et al., 2020). They are also frequently used as a partial replacement for rapid embankment construction on a soft soil layer. This is because stone columns have a high degree of permeability and stiffness values, which allow for faster dissipation of excess pore pressure and help accelerate consolidation in the soft soil layer (Tan et al., 2008).

The stone column technique has gained significant attention as a ground improvement method for rapidly constructing embankments on soft soil (Yoo, 2010). Stone columns are an effective method to enhance soft soils due to two crucial factors. Firstly, they involve using a stiffer column material, such as crushed stones or gravel placed within the soft soil. Secondly, when a stone column is installed, it densifies the surrounding soft soil, followed by consolidation (Guetif et al., 2007).

Many scholars have proposed using stone columns to increase soil stability and reduce settlements in soft soils (Alkhorshid et al., 2021; Deb et al., 2011; Bailappanavar et al., 2021; Choobbasti et al., 2011; Belayneh, 2020; Etezzad et al., 2015; Almeida et al., 2018; Muzammil et al., 2018). Using stone columns and basal geogrids can reduce subsoil deformations and improve the stability of embankments (Ghorbani et al., 2021).

This study examines the behavior of an embankment constructed on soft soil reinforced with multiple columns. The analytic method allows us to estimate settlement and stress distributions at any location beneath the embankment. The approach was the axisymmetric-to-plane strain conversion method proposed by (Ng and Tan, 2015). A road embankment supported by several stone columns was analyzed using the 2D finite element method. The axisymmetric column diameter was considered as the plane-strain column width and kept the center-to-center distance of the columns constant. Additionally, the equivalent plane-strain permeability was calculated based on correlation equations.

2 PLAIN STRAIN CONVERSION

(Ng and Tan, 2015) proposed two strategies for transforming the equivalent plane-strain model from the axisymmetric unit-cell model. Both approaches retain the distance between stone columns, while introducing an expanded shear wall for each stone column. In the first approach, the soil permeability varies according to the correlation equations, and the wall's width is the same as the stone column's diameter. Nevertheless, for the second approach, the wall's width is minimized, while the permeability of the soil materials remains unchanged. For modeling linear-elastic material, both techniques yield reasonably realistic long-term consolidation settlements. In this study, the first method proposed by (Ng and Tan, 2015) is applied to analyze an embankment on stone column-reinforced soft soil.

It is feasible to consider the diameter of the axisymmetric column (r_c) to be equal to the plane-strain column's width (b_c) as shown in Fig. 1 (a and b), i.e.:

$$b_c = r_c \quad (1)$$

As a result, the equivalent plane-strain width (B) could be the same as the radius of the drainage zone (R), i.e. :

$$R = B \quad (2)$$

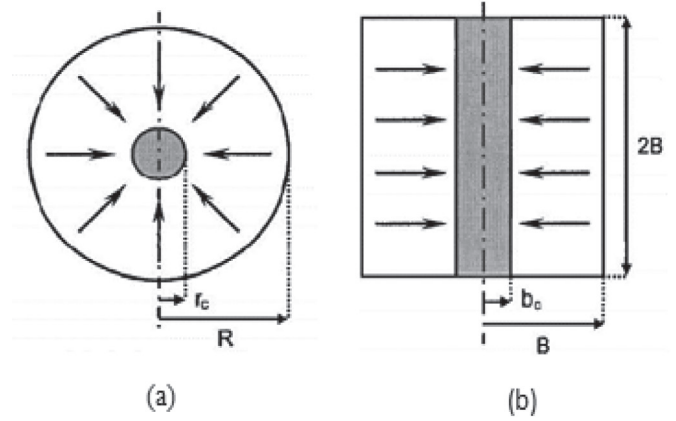


Fig. 1 Axisymmetric to plain strain conversion (Tan et al., 2008) ; (a) axisymmetric (b) Plain strain

This method enables a smooth transition from an axisymmetric to a plane-strain conversion. The plane-strain material's properties must be modified to consider the geometric modifications. The following equation, which matches the stiffness of the column-soil composite, provides the plane-strain material's stiffness.

$$E_{c,pl} a_{s,pl} E_{s,pl} (1 - a_{s,pl}) = E_{c,ax} a_{s,ax} + E_{s,ax} (1 - a_{s,ax}) \quad (3)$$

where, E_s and E_c are elastic moduli of the surrounding soil and the column material, respectively. Additionally, the subscripts *ax* and *pl* denote axisymmetric and plain-strain conditions, respectively. The area replacement ratio is given by $a_s = A_c / (A_c + A_s)$, where A_s and A_c are cross-sectional areas of the surrounding soil and column, respectively.

The following analytical equation is used to match the soil permeability as derived by (Tan et al., 2008):

$$\frac{k_{h,pl}}{k_{h,ax}} = \frac{F(N)_{pl}}{F(N)_{ax}} \left[\frac{m_{vs} m_{vs} (1 - a_s)}{m_{vs} (1 - a_s) + m_{vs} a_s} \right]_{pl} \left[\frac{m_{vc} (1 - a_s) + m_{vs} a_s}{m_{vs} m_{vc} (1 - a_s)} \right]_{ax} \frac{B^2}{R^2} \quad (4)$$

where k_h = coefficient of soil permeability in the horizontal direction:

$$F(N) = \left[\frac{N}{N^2 - 1} \right] \ln(N) - (3N^2 - 1)/(4N^2) \quad (5)$$

Diameter ratio for axisymmetric condition:

$$N = R/r_c \quad (6)$$

Diameter ratio for the plane-strain condition:

$$N = B/b_c \quad (7)$$

$$m_{vs} = (a_{vs}) / (1 + e_s) \quad (8)$$

$$m_{vc} = \left(\frac{a_{vc}}{1 + e_c} \right) \quad (9)$$

where a_{vc} and a_{vs} are coefficients of the compressibility of the column and the surrounding soil, respectively, e_c and e_s are the void ratios of the columns and the surrounding soil, respectively. It is anticipated that the plane-strain coefficient of the soil

permeability in the vertical direction, k_v , pl , will have little effect on the overall consolidation rate because the movement of water in the soil is mainly radial or horizontal, i.e., $k_{v,pl} = k_{v,ax}$.

3 VALIDATION OF THE MODEL

In this study, the findings of the field measurement results were used to verify the reliability of the 2D finite element model. The settlement of the actual embankment at selected points and the excess pore pressures developed at various spots in the soft soil was measured, as reported by (Tan et al., 2008). As depicted in Fig. 2, the pore pressures were recorded at the *A*, *B*, and *C* locations, and the settlements were measured at the settlement plates 1 and 2. In addition, the embankment had a side slope of 1V-2H, a height of 1.8 m, and a base width of 40 m. The 80 cm diameter stone columns were arranged in a square grid with 2.4 m center-to-center spacing, extending 6 m below the ground's surface. In order to facilitate construction activities on the soft soil and enhance drainage, a layer of dense, granular soil 1 m thick was added on top of the soft clay.

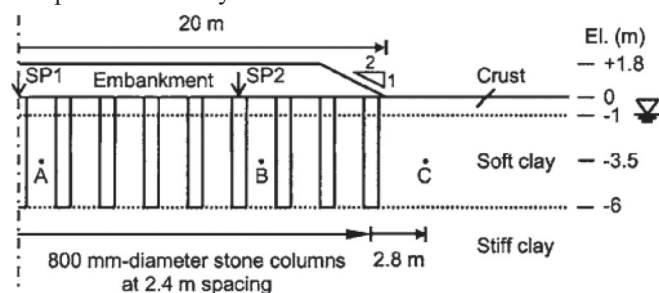


Fig. 2 An actual embankment in Malaysia, 2003 (Tan et al., 2008)

The geotechnical engineering program, Finite Element Package (Plaxis 2D V21), was used to analyze the behavior of an embankment on soft soil. Several researchers have extensively used it to analyze various geotechnical problems (Rajesh and Jain, 2015; Lo et al., 2010; Hataf et al., 2020). As stated in Tab. 1, the input parameters for the soil layers, stone columns, and embankment fill were adopted from a previous researcher. The Mohr-Coulomb material model was used for the road embankment, soil layers, and stone columns. The Mohr-Coulomb model is a linear elastic-plastic model that requires five input parameters to describe stress-strain behavior. This model is frequently used because it has a simple formulation and requires less input data from basic tests, making it more applicable than other models. Nevertheless, a linear elastic material property was considered for the basal geosynthetic material.

Tab. 1 Input parameters for the numerical analysis (After [(Ng and Tan, 2015)])

Material	γ_{unsat} (kN/m ³)	γ_{sat} (kN/m ³)	ν'	E (kPa)	k_h (m/s)	k_v (m/s)	c (kPa)	ϕ (°)
Stone column	19	20	0.3	8,670	$1.16 \cdot 10^{-4}$	$1.16 \cdot 10^{-4}$	5	40
Stiff clay	18	20	0.3	40,000	$3.47 \cdot 10^{-9}$	$1.16 \cdot 10^{-9}$	3	30
Soft clay	15	15	0.3	1,100	$1.68 \cdot 10^{-9}$	$1.16 \cdot 10^{-9}$	1	20
Crust	17	18	0.3	15,000	$3.47 \cdot 10^{-7}$	$1.16 \cdot 10^{-7}$	3	28
Road Embankment	18	20	0.3	15,000	$1.16 \cdot 10^{-5}$	$1.16 \cdot 10^{-5}$	3	33

The model was meshed with a fine-mesh using triangular elements with 15 nodes, as shown in Fig. 3, and divided into 619 elements and 5127 nodes. The numerical analysis requires only half of the embankment section to be considered due to the axial symmetry at the center. To verify the model, the embankment's geometry, settlement plate positions, pore pressure locations, clay layer dimensions, and stone column dimensions are identical to those reported by (Tan et al., 2008). The embankment was constructed in three phases, with each step requiring three days to place the 60 cm-thick fill. As a result, the embankment took nine days to finish and reached a height of 1.8 m. After the loading, the embankment was entirely constructed; a consolidation study was performed; and the computations were continued until the excess pore water pressure had completely dissipated. The flow boundary condition was open at the right boundary and at the top of the crust layer and closed at the left boundary and the bottom of the stiff clay layer. The drainage along the left boundary was closed because half of the construction was simulated.

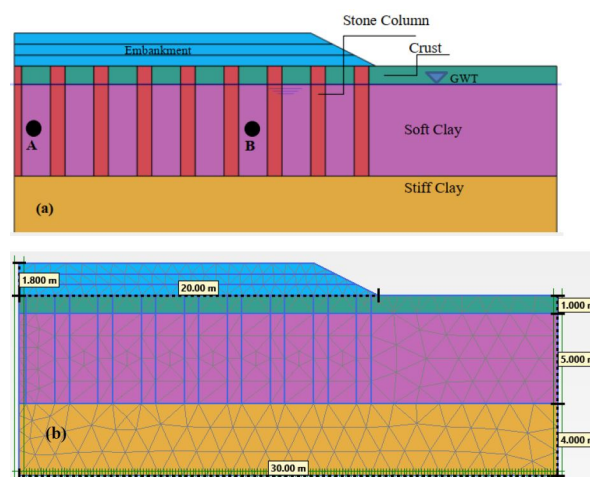
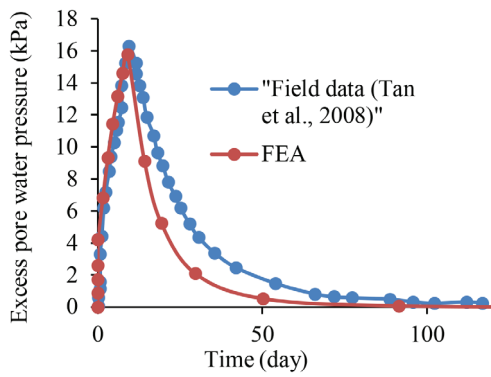
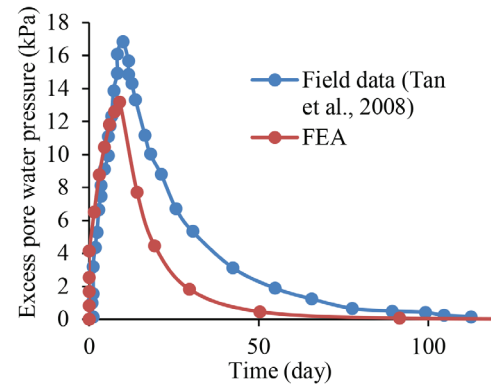


Fig. 3 Validation of the model; (a) Numerical model used for the analysis (b) Generation of the mesh

The excess pore water pressure values from the field measurements at sites A and B, see Fig. 3, were compared against the numerical analysis to demonstrate the model's viability. As demonstrated in Fig. 4, the outcomes of the numerical results showed good agreement with the field measurement results. Therefore, the computed numerical model is valid for a further analysis of the different parameters. Fig. 5 depicts the variations of the simulated and measured settlements over time for a position beneath the centerline of the embankment at (0, 0). It is evident that the numerical analysis follows the trend of the results measured and makes relatively accurate predictions.



(a) Measurements at A



(b) Measurements at B

Fig. 4 Field and FEA of the pore pressures at points A and B

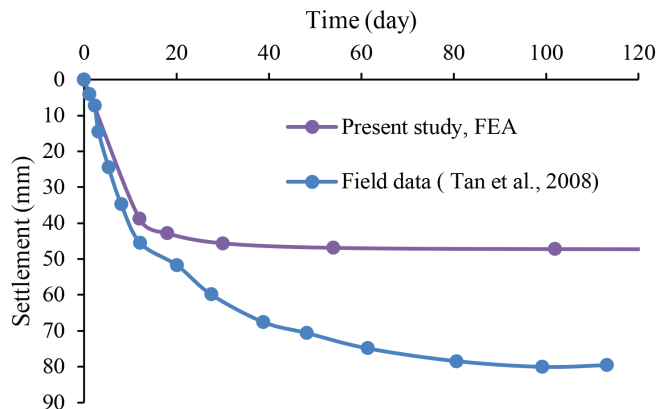


Fig. 5 Settlement variations at point (0,0) for the numerical analysis and field measurements

4 PARAMETRIC ANALYSIS

Variables such as the area replacement ratio (a_s), length of the stone columns (L_s), and basal geogrid stiffness (J) were considered when analyzing the road embankment. The values of each parameter are given in Tab. 2. The embankment was built in three stages, each requiring three days to construct the 1.2 m thick fill material. As a result, the embankment took nine days to finish and reached a height of 3.6 m. Following the completion of loading, i.e., the complete construction of the embankment, a consolidation study was carried out, and the calculations were continued until the excess pore water pressure had completely dissipated.

The embankment had a base width of 50 m, a height of 3.6 m, and a side slope of 1V-2H. The actual embankment's stone column diameter, spacing, and slope remained unchanged. The stiff clay was 9 meters deep, as illustrated in Fig. 6, and the total depth of the soil strata in this inquiry was 15 meters. The numerical model shown in Fig. 6 is a new model adopted by altering several geometric dimensions from the initial numerical model used for the validation. The height of the embankment was increased to 3.6 m to improve its performance and clarify the effect of the geogrid on the parametric analysis. Moreover, the mesh's coarseness impacted the pore water pressure and consolidation settlement. Since fine mesh has a higher degree of accuracy (Tan et al., 2008), it was chosen for the numerical analysis of this study.

Tab. 2 Variables for the parametric analysis

Variables		Units
Area replacement ratio, a_s (%)	10% (2.3)*, 15% (1.8)*, 20% (1.6)*	m
Length of stone column	$0.5 H_s$, $0.75 H_s$, H_s *	m
Basal geogrid stiffness	3500	kN/m

* Center-to-center column spacing, H_s *Height of soft soil layer

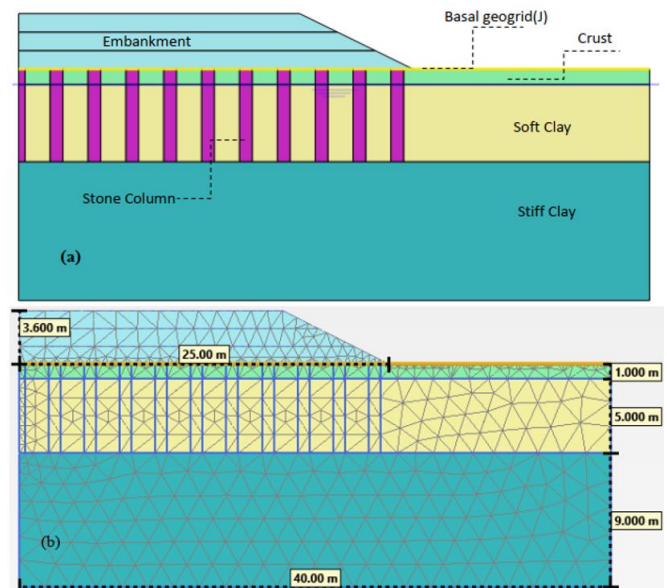


Fig. 6 The numerical model adopted for the parametric studies and mesh generated

5 RESULTS AND DISCUSSIONS

5.1 Effect of the area replacement ratio (a_s)

The relationship between the column area (A_c) and its area of influence (A_E) is known as the area replacement ratio ($a_s = A_c/A_E$). Equation (10) is used to compute the area of inf-

fluence A_E . The values of the diameter of the area of influence (d_e) are taken as $1.05s$ (for a triangular mesh), $1.13s$ (for a square mesh), and $1.29s$ (for a hexagonal mesh), where “ s ” is the spacing between the columns (Almeida et al., 2018).

$$A_E = \pi \frac{d_e^2}{4} \quad (10)$$

The stone column spacing (s) of 2.4 m, which is similar to the actual embankment, was used for the finite element analysis on a square mesh. The settlement of the embankment decreases as the area replacement ratio increases, as shown in Fig. 7. After 100 days, the settlement measurements were as follows: 54.28 mm, 36.20 mm, and 33.12 mm for 10%, 15%, and 20% area replacement ratios, respectively. The bearing capacity of a single or group of stone columns increases, and the settlement reduces with increasing the area replacement ratio (Fattah et al., 2017; Nav et al., 2020).

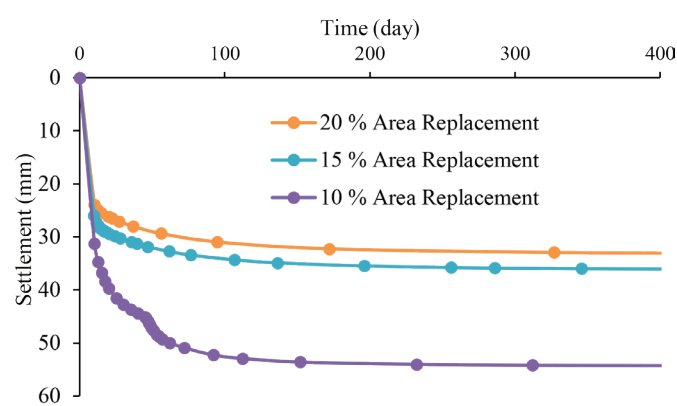


Fig. 7 Effects of the area replacement ratio on the settlement

5.2 Effects of the length of the stone columns (L_s)

The length of the stone columns varies, based on the thickness of soft clay (H_s), as indicated in Tab. 2. The diameter and spacing remained constant at 0.8 and 2.4 meters, respectively. A stone-column-reinforced embankment was analyzed to avoid failure. The length of the stone columns affected the settlement of the road embankment, as depicted in Fig. 8. It is evident from the figure that changing the length of the stone columns affected both the heave at the embankment toe and the settlement beneath the embankment. For instance, the maximum settlement would be dropped from 150 mm to 57 mm if the column's length was increased from $0.5H_s$ to H_s . The bearing capacity was improved, and the settlement was reduced when the length of the stone column increased (Ali et al., 2011).

Additionally, Fig. 8 demonstrates how a longer stone column length speeds up the consolidation process and decreases the time needed for the ultimate settlement. The dissipation time, for instance, is decreased from 588 to 45 days by increasing the stone column's length from $0.5H_s$ to H_s . Furthermore, increasing the length of the stone columns from $0.5H_s$ to H_s reduces the maximum pore pressure from 40 to 19 kPa (a 52.5% decrease), as shown in Fig. 9.

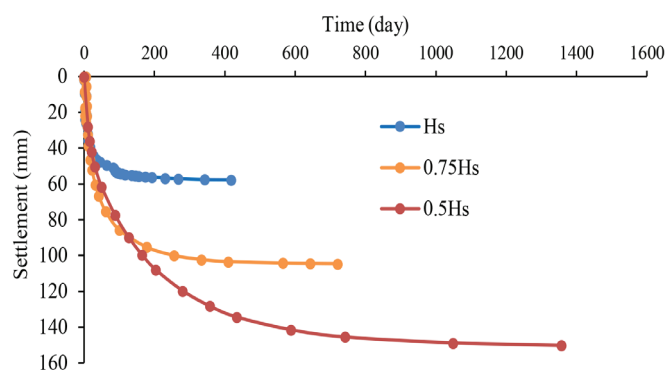


Fig. 8 Effect of variations of the length of the stone column on settlement

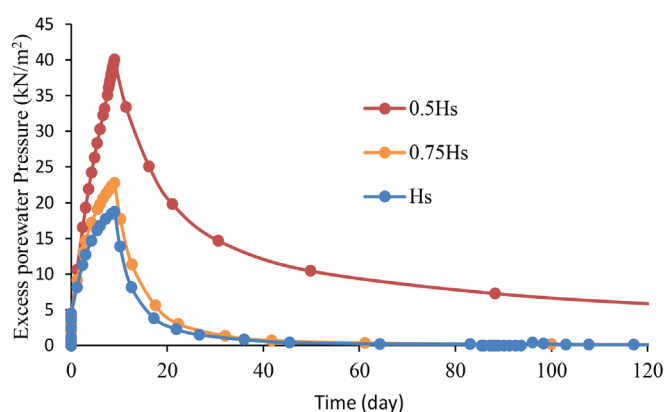


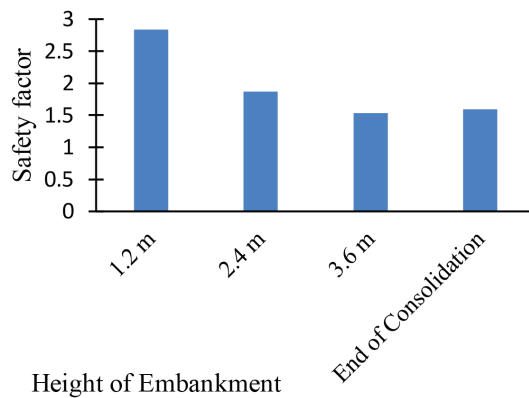
Fig. 9 Variation of the pore water pressure in the middle of the soft soil layer

5.3 Effects of the stiffness of the basal geogrid (J)

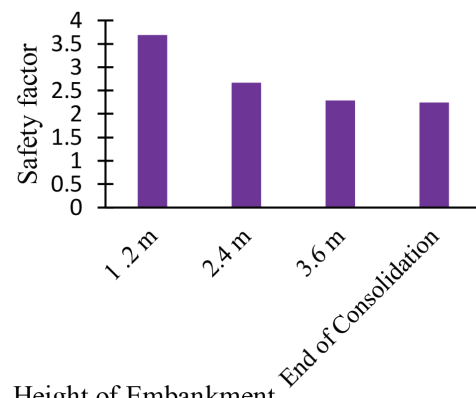
A strength reduction method was carried out for determining the safety factor during the three successive stage constructions. The height of the embankment is 3.6 m, and it is constructed in three consecutive stages. Each stage was allowed to consolidate for three days, and a safety analysis was conducted at the end of each construction stage. As depicted in Fig. 10, the safety factor decreases as the stage construction increases. The safety factor analysis was conducted for an untreated and a basal geosynthetic-treated embankment having a stiffness value ($J = 3500$ kN/m). Basal geogrid-reinforced embankments have a considerably higher safety factor than unreinforced embankments for each stage of construction and the global safety factor analysis, as demonstrated in Fig. 10. Using a basal geogrid reinforcement in embankments helps to decrease lateral displacements and improves overall stability (Liu et al., 2007).

6 CONCLUSIONS

The behavior of an embankment reinforced with a basal geogrid and supported by stone columns was studied using the plain strain conversion method. Following the model's validation, parametric studies were conducted to determine the effect of the area replacement ratio, stone column length, and basal



(a) Reinforced stone column



(b) Reinforced stone column and basal geogrid

Fig. 10 Safety factor for stage constructions of the embankment

geogrid's stiffness on the embankment's behavior. The following conclusions can be drawn from the numerical analysis:

- The settlement reduces when the area replacement ratio increases. For example, the settlement decreases from 54.28 mm to 33.12 mm after 100 days when the area replacement ratio rises from 10% to 20%.
- The increase in the length of the stone column notably reduces settlement and excessive pore water pressure. As the length of the stone column increases from 0.5 times the height of the embankment to the full height of the embankment, there is a decrease in excess pore water pressure from 40 kPa to 19 kPa. It is worth noting that the length of a stone column can affect the settlement beneath the embankment and the heave at the embankment toe. By increasing the column length from $0.5H_s$ to H_s , the maximum settlement of a road embankment built on soft soil was reduced from 150 mm to 57 mm.

- Based on the results of the stability analysis, it was observed that using basal geosynthetics during all the construction phases led to an increase in the embankment's stability. The safety factor at the end of consolidation increased from 1.6 to 2.3 due to reinforcing the road embankment with the basal geogrid.
- In general, the increase in the area replacement ratio, the length of the stone column and the reinforcement of the road embankment at the bottom reduces the settlement and increases the stability of the embankment. This study was conducted for soft soil with a maximum depth of 6 meters. Further studies are recommended for deeper soft soils.

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EXPERIMENTAL CONTRIBUTION TO STUDY THE PHYSICO-MECHANICAL AND THERMAL PROPERTIES OF LIGHTWEIGHT CELLULAR CONCRETE PREPARED WITH DIFFERENT TYPES OF SAND AND WASTE MARBLE POWDER

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Abstract

Algeria is currently undergoing an evolution in civil engineering, which has resulted in a higher use of aggregates, especially sand. This study contributes to the development of locally produced materials. Its goal is to explore how the type of sand and waste marble powder (WMP), which can be used as a partial replacement of cement and aluminum powder (Al) with different percentages as an expansive agent, can affect physico-mechanical properties such as the bulk density and mechanical strength at 28 and 90 days, as well as the thermal conductivity of lightweight cellular concrete. Non-autoclaved cellular concretes (CC) were investigated based on three types of sand, i.e., sea sand (SS), river sand (RS), and waste marble sand (WMS); a hydraulic binder (a mix of cement and waste marble powder), and aluminum powder. The experimental results indicate that the nature and particle size distribution of the sand used had an impact on the properties of the CCs. Furthermore, it can be seen that the increased aluminum contents decrease the density, mechanical strength, and thermal characteristics of the CCs. Using 10-15% WMP as a cement substitute improves the mechanical strength, offers better thermal conductivity values, and contributes to sustainable development by developing a new class of environmentally friendly cellular concrete.

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Key words

- Lightweight cellular concrete,
- Sand,
- Aluminum powder,
- Thermal conductivity,
- Physico-mechanical properties.

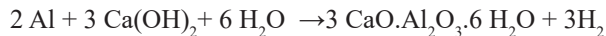
1 INTRODUCTION

In recent decades, the improvement of structures and structural building works has resulted in significant energy use. It has become critical to consider all of the potential outcomes of waste reuse and recovery in the context of manageable improvements and environmental preservation, because the need to recoup materials is taken into account on the one hand, while, on the other hand, there is a need to protect the environment. Like some other European and African countries, Al-

geria is experiencing a severe shortage of structural materials, particularly cement and sand (Belachia et al., 2011; Akbulut et al., 2007; Binici et al., 2008). Similarly, ongoing research has focused on the valorization of alternative materials to manufacture low-cost and durable structural materials (Cisse et al., 2000). In terms of valuing Algeria's many assets (natural and industrial) and their possible misuse, lightweight concrete is intriguing from both financial and environmental standpoints. A few scientists have shown that the removal of wellsprings of regular fine totals, such as river sand, poses a risk of debase-

ment that is harmful to nature (Sun et al., 2016; Dias, et al., 2008). Different specialists have recently become interested in the use of marble powder as an alternative material (Agarwal et al., 2006). According to these findings, the presence of marble powder in a cementitious grid improves compressive quality at an early age (Messaudene et al., 2011). The effect of marble powder as a filler has been studied (Topcu et al., 2009). The mechanical strength of cement decreases; however, its handling is unaffected for a large amount of marble (Corinaldesi et al., 2010). According to the results, replacing Portland cement with 10% marble powder improves the compressive strength of concrete by more than 50 MPa at 28 days (Messaudene et al., 2011).

Cellular concrete is a cost-effective alternative for producing large-scale lightweight materials and components. It is composed of various components such as fine sand (silica sand powder), cement, hydrated lime, water, and an expansive agent (Ramamurthy et al., 2009). The method of autoclaving is frequently employed in the manufacturing process to generate autoclaved aerated concrete (AAC), by which millions of tiny non-connecting cells are created when aluminum powder is combined with calcium hydroxide in the presence of water resulting from the hydration of the cement/lime to form hydrogen gas (Jerman et al., 2013; Damene et al., 2018); it generates large-scale pores due to the extension of the mixture, with the pore space accounting for 60 to 90 % of the total volume (Narayanan et al., 2000); however, some hydrogen gas is ineffective in providing pores because of the high amount of aluminum powder. The chemical reaction that occurs as follows:



Nevertheless, the cost of producing AAC is still highly expensive because it requires an autoclave and uses a lot of energy. The porosity of cellular concrete is affected by several parameters, including the composition of the mix design, foam agents, and curing types. The findings suggest that the porosity of aluminum powder increases up to 0.4 % before decreasing to between 0.6 - 0.8%. The mechanical strength of cellular concrete increases as its porosity decreases (Guglielmi et al., 2010). The pore size and microstructure affect properties such as the cc's strength, durability, heat transmission, and moisture transport (Narayanan et al., 2000; Torquato, et al., 2002). Various studies (Lian et al., 2011; Boutin, 1996) discovered that the CC's compressive strength is proportional to its density and that it increases as the density increases (Mugahed et al., 2015). The latter depends on many variables, such as the rate of the foaming agent, which decreases the density as it increases, because it creates air voids, the w/c proportion, the method of curing, which can be non-autoclaved or autoclaved (Narayanan et al., 2000), and the type of sand. The particles damage the strength of the cellular concrete because its expansion affects the pore size in the paste (Nambiar et al., 2006). Indeed, some researchers (Cabrillac et al., 2006; Damene et al., 2018) studied lightweight cellular concrete and concluded that the use of hydrated lime as a partial substitute for cement and aluminum content tended to increase the total porosity of the matrix and decrease both the thermal conductivity and compressive strength. (Ramamurthy et al., 2009), and (Richard et

al., 2013) reported that foamed concrete has various features such as a low density, which improves its thermal performance due to its microstructure.

With the aim of developing local materials, our research sought to investigate the impact of different types of sand on the physico-mechanical and thermal properties of cellular concrete without requiring any autoclave treatment by a lightening process in a Sand/Binder mineral paste by a chemical reaction with aluminum powder that confers a cellular structure on the material. A secondary purpose was to determine the effects of waste marble powder content as a partial replacement for different aluminum dosages on physico-mechanical behavior such as the bulk density and mechanical strengths at 28 and 90 days, as well as the thermal conductivity of cellular concrete.

2 EXPERIMENTAL DETAILS

2.1 Materials

The samples for the binder are a mixture of cement and waste marble powder, which was substituted in the cement mass (0%, 10%, 15%, and 20%). The cement used was a CEM I 42.5 ordinary Portland type, manufactured by the LE KARROUBA factory located in Tunisia, according to the European Standards EN-197/1. Table 1 lists its chemical composition and its physical and mechanical properties. Fines must be added to the concrete to increase its compactness by completing the granular framework, thereby minimizing the amount of expensive cement in the binder. The waste marble powder (WMP) is a fine that comes from a marble quarry in the Skikda region (Algeria); it was prepared by sifting it (in a sieve opening of 80 μm) and crushing the waste marble sand used. Its characteristics are given in Table 2. Three types of sand were used for making the cellular concrete, i.e., sea sand (SS), obtained from the region of Jean dark located in Skikda; river sand (RS), obtained from the region of Ibn ziad located in northwestern Constantine; and waste marble sand (WMS), from the marble quarry of Fil-fila, situated 25 km to the east of the region of Skikda. The sands have maximum grain diameters of 1 mm, 4 mm and 3 mm, respectively. As established according to standard NF P18-560, these sands have a continuous particle size distribution. RS has the highest fineness modulus, i.e., (3.06), indicating that it is the coarsest sand, whereas SS and WMS have fineness moduli of 2.10 and 2.03 respectively, and are finer sands. The "SS and RS sand equivalent" values are measured according to the NF P 18-598 standard, which shows that SS and RS are clean sands, while the "WMS sand equivalent" is 65%. This value remains the value recommended for concrete and mortar (Belhadj et al., 2014). Table 3 and Fig. 1 show the physical properties and the grading curves of these various sands. Aluminum powder (Al) with a fineness of 100 μm commercialised by SIALCHIM Company and a purity of 99% and a molar mass of 26.98 $\text{g} \cdot \text{mol}^{-1}$ was used as the expanding agent in the lightening process. The water (W) used for mixing the CC constituents was untreated tap water. The PH was close to 7.34 to $T^\circ = 13.2^\circ\text{C}$ when tested on a regular basis.

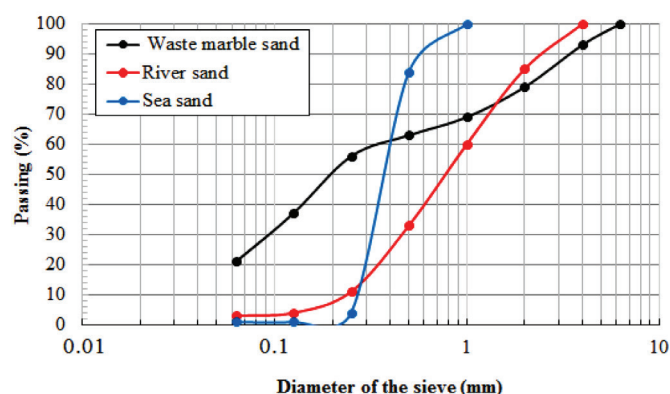


Fig. 1 Grading curve of the sands

2.2 Mix proportions

All of the mixtures were made using an S/B ratio of 3/2. There were 60 compositions produced in total, which were proportioned and divided into three groups using the three distinct sands, i.e., SS, RS, and WMS, each with twenty mixes. Only cement was used as a binder in the composition of the control mix with (WMP/B) = 0% and (A/B) = 0%. Whilst the cement was partially substituted with WMP at ratios of (10%, 15%, and 20%), varying percentages of the expanding agent (A) by the weight of the binder (from 0 to 1 %, with 0.25% increments) were included for each mixture.

The amount of the W/B ratio was defined for each mix design of the different sands by studying the flow time results of the fresh concrete obtained using an LCPC maniabilimeter in order to satisfy the workability requirement and the RILEM standards for lightweight concrete (RILEM, 1993). The W/B ratio of about 50% for SS, 55% for RS, and 60% for WMS (see table 4) shows the proportions of all the CC mixtures.

Tab. 1 Properties of the cement

Properties	Value
Initial setting time (min)	210
Final setting time (min)	264
Specific gravity (g/cm ³)	3.2
Blaine fineness (cm ² /g)	38.69
Compressive strength at 2 days (MPa)	21.9
Compressive strength at 14 days (MPa)	37.6
Compressive strength at 28 days (MPa)	50.10
Flexural strength at 2 days (MPa)	4.68
Flexural strength at 7 days (MPa)	7.01
Flexural strength at 28 days (MPa)	8.37
C ₄ AF (%)	14
C ₃ A (%)	3.14
C ₃ S (%)	67.54
C ₂ S (%)	8.83

Tab. 2 Properties of WMP

Properties	WMP
Color	White
Specific gravity (g/cm ³)	2.8
Blaine fineness (cm ² /g)	58.04
CaO	59.54
Al ₂ O ₃	1.06
Fe ₂ O ₃	0.13
SiO ₂	1.76
Mg O	1.03
Na ₂ O	0.04
K ₂ O	0.03
SO ₃	0.01
Cl ⁻	-

Tab. 3 Physical properties of the sands used

Properties	Sand type			
	Unit	SS	RS	WMS
Absolute density	(kg/m ³)	2580	2610	2600
Sand equivalent	(%)	95	77	65
Fineness modulus	(%)	2.1	3.06	2.03
Fines content	(%)	1	3	21

Tab. 4 Mix proportions of the CC mixtures

CC mixture	B=WMP+ C		% S/B	% AI/B	% W/B
	% WMP/B	% C /B			
Group 1					
SSCC	0	100	150	0	0.50
SSCC WMP 10%	10	90		0.25	
SSCC WMP 15%	15	85		0.50	
SSCC WMP 20%	20	80		0.75	
Group 2					
RSCC	0	100	150	0	0.55
RSCC WMP 10%	10	90		0.25	
RSCC WMP 15%	15	85		0.50	
RSCC WMP 20%	20	80		0.75	
Group 3					
WMSCC	0	100	150	0	0.60
WMSCC WMP 10%	10	90		0.25	
WMSCC WMP 15%	15	85		0.50	
WMSCC WMP 20%	20	80		0.75	



Fig. 2 volumetric expansion in the samples and distribution of the voids in the mortar matrix

2.3 Mixing and moulding procedures

Regarding the cellular concretes studied, the solid elements, i.e., the “waste marble powder + cement” binder and sand, were first combined for two minutes (EN 196-1). An amount of water was added; then, the concrete was mixed for another two minutes at a low speed, followed by a pause in order to homogenize the mixture by scraping the bowl’s bottom and sides. Another two minutes of mixing at a high speed followed. The expansive agent was introduced at a low speed for one minute, which was then followed by one minute at a high speed (Goual et al., 1999). All the mixtures were poured into (40 x 40 x 160 mm³) prismatic moulds and stored without a cover in a storage chamber ($T = 20 \pm 2^\circ \text{C}$ and $\text{RH} = 95 \pm 3\%$). After 24h, the expanded part of the specimens was sawed before demoulding and stored in a climatic room ($T = 20 \pm 2^\circ \text{C}$ and $\text{RH} = 95 \pm 3\%$) until testing. Fig.2 indicates the volumetric expansion in the samples and distribution of the voids in the mortar matrix.

2.4 Testing procedures

The physico-mechanical and thermal performances of the cellular concrete were determined by the dry bulk density ρ_{app}

according to the ASTM-C134 standard by dividing the dry weight by the apparent volume of the test specimens using the following relation :

$$\rho_{app} = \frac{D_w}{L \times W \times H} \quad (1)$$

where D_w , L , W , and H , are the dry weight, length, width and height of the samples respectively.

The compressive strength tests were measured according to NF EN 196-1 by carrying out on the specimens with dimensions of 40 × 40 mm test sections on short and long terms of “28 and 90 days” (Fig. 3) with a maximum compression force of 2400 N/s ± 200 N/s. The compressive strength results on each curing day corresponded to the average of the three measurements taken on the samples of each combination of the different types of sand used to make the cellular concrete.

Thermal conductivity λ and diffusivity are two standards used to measure the insulating qualities of building materials. These are determined by the components’ inherent qualities, the material’s microstructure, and the storage circumstances (the role of water) (RILEM, 1970). The thermal diffusivity (α), of a material is defined as the speed at which heat propagates by conduction in a medium. A material with a lower thermal diffusivity value will have a slower heat transfer rate throughout its thickness, resulting in higher insulation properties. It is calculated by dividing the thermal conductivity by the specific heat and density (m²/s), as demonstrated in equation (2).

$$\alpha = \frac{\lambda}{\rho C_p} \quad (2)$$

Our interest in this study is the determination of the thermal conductivity measurements of the materials developed using the Hot Disk TPS 1500 method, which is a Hot Disk sensor in the shape of a double spiral consisting of an electrically conducting pattern. This spiral is sandwiched between two thin insulating sheets (Fig. 4(a)). When making a measurement, the Hot Disk sensor is placed between the two plane samples to be studied in dimensions of (40 x 40 x 20 mm³). A chucking device was used to provide proper contact between the various components and to prevent the two samples from moving according to the experimental assembly shown in Fig. 4(b). The sample should be larger than the sensor diameter so as to avoid impact from the sample’s outside boundaries and to offer reliable values for the thermal conductivity and diffusivity. The power heat flux was kept constant at 0.55 W, and



Fig. 3 Simple compression instrument

the overall duration of the transitory regime varied from 40 to 80 seconds in order to acquire accurate results.

When measuring the thermal conductivity of the sample surrounding the TPS-element, the TPS method uses a conducting pattern with a heat capacity concurrently as a heating element and a temperature sensor (Gustafsson, 1991; Bouguerra et al., 1997). The equation can be represented as the following time-dependent resistance of the TPS-element during the transient recording:

$$R(t) = R_0 (1 + \alpha \Delta T(\tau)) \quad (3)$$

where $R(t)$ is the resistance of the TPS element at time (t); R_0 is the resistance of the TPS element at time zero; α is the temperature coefficient of the resistivity (TCR); and $\Delta T(\tau)$ is the mean value of the increase in the TPS-element's temperature. The graph of the experimental temperatures T_0 recorded at each second is then obtained. The thermograph recording the increase in temperature in the TPS probe revealed that a range of measuring points between 60 and 160 of each series of measurements could be used to evaluate the thermal parameters.



Fig. 4 (a) Hot Disk sensor

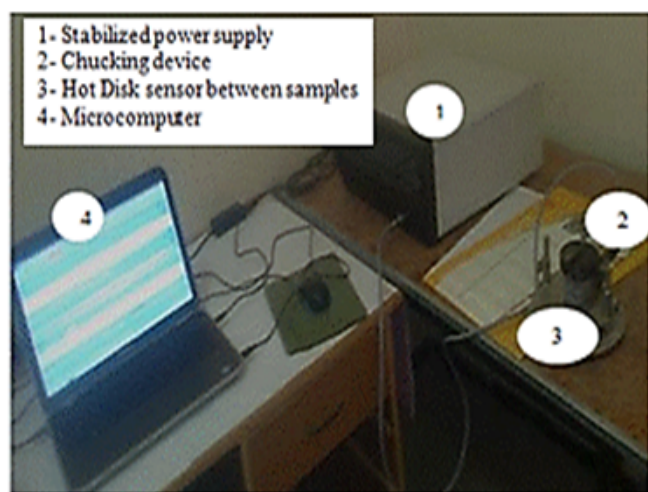


Fig. 4 (b) Experimental assembly used in the Hot Disk technique

Fig. 4 Experimental assembly used in the Hot Disk technique

3 RESULTS AND DISCUSSION

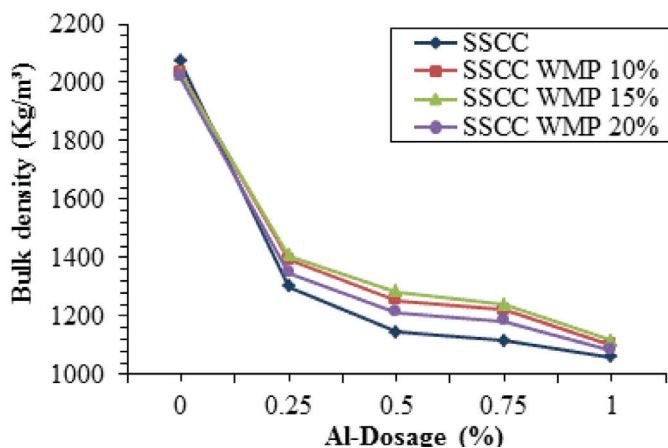
3.1 Bulk density

The density of cellular concrete is one of its most important features. Based on this property, it is possible to classify the type of concrete and identify its field of application. The experimental results of the dry bulk density of the CCs developed as a function of the different percentages of the aluminum powder ranging from 0 to 1 % for the different WMP contents are presented in (Fig. 5). The curves show that the bulk density of cellular concrete made with the various types of sand decrease significantly as the aluminum content increases due to the decreasing solid content and that the voids created by the hydrogen released by the response of the aluminum lighten the material (Nambiar and Ramamurthy, 2006 a,b) and lead to high variations in the density (Ramamurthy et al., 2009). Furthermore, for the various aluminum powder percentages, the decrease in bulk density is more important for the reference mixtures without WMP (SSCC, RSCC, and WMSCC). In terms of the impact of the WMP content, the results indicate that the bulk density increases at the level [0–10%] of the WMP content for the compositions of the WMSCC developed and at the level [0–15%] of the WMP content for the compositions of the SSCC and RSCC developed, which also had the maximum compressive strength of the CC_s. The addition of WMP had a filling effect, which made the matrix more compact and dense (Vardhan et al., 2019). The decreasing rates of the bulk density vary from 48.7 % for the SSCC composition without WMP to 44.9 % for the composition containing 15% of WMP (SSCC WMP 15%) when the aluminum rate percentage is between 0 to 1 % (Fig. 5(a)). The rate of decay in the dry bulk density decreased from 47.8 % for the RSCC to 43.6 % for the RSCC WMP 15% compositions as the aluminum content varied from 0% to 1% (Fig. 5(b)). This parameter varied for the WMSCC from 48.2 % to 45.15 % for the WMSCC WMP 10% compositions for the same range of the aluminum dosage (Fig. 5(c)). It can be seen that the densities reached a specific threshold (0.5% of the aluminum content in this case study), varied slightly, and then became constant for the remaining percentages of aluminum. These findings confirm those of (Cabrillac et al., 2006; Damene et al., 2014).

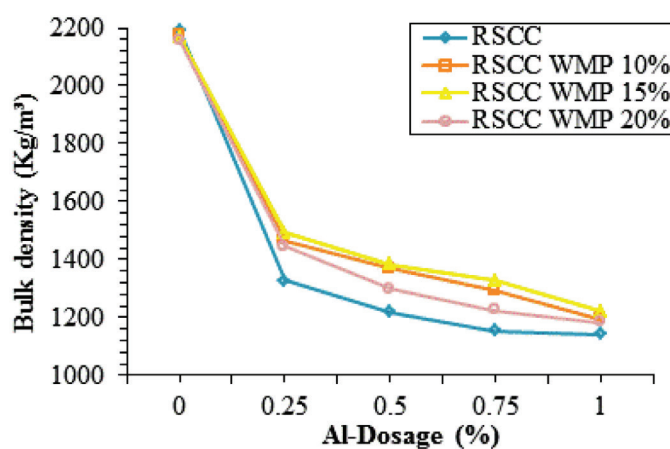
Regarding the effect of the type of sand used, the findings also revealed that the changing of the type of sand is essential for reaching the desired density in cellular concrete. The results show that the river sand resulted in the highest density, followed by the waste marble sand, while the sea sand resulted in the lowest density when the aluminum content varied from 0 to 1 %. This is mainly due to the three sands' distinct grain size distributions and their different physical properties (shape, density, etc.).

3.2 Compressive strength

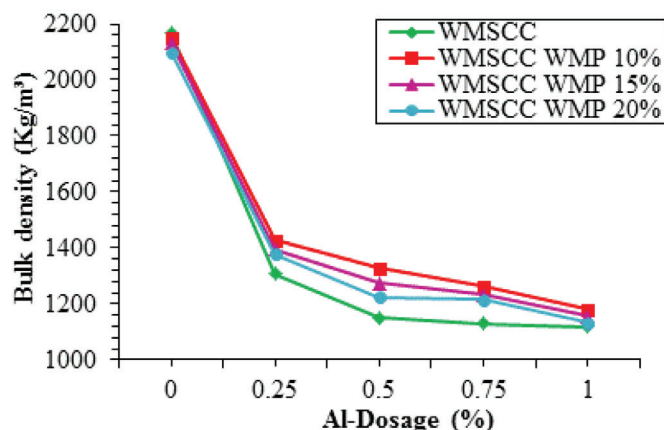
Several factors affect compressive strength, which is directly correlated with density (Lian et al., 2011). Fig.6 illustrate the variations of compressive strength as a function of the dosage of aluminum for different substitution ratios of the waste marble powder content (WMP/B) at the various cur-



5(a) Sea sand cellular concrete (SSCCs)



5(b) River sand cellular concrete (RSCCs)



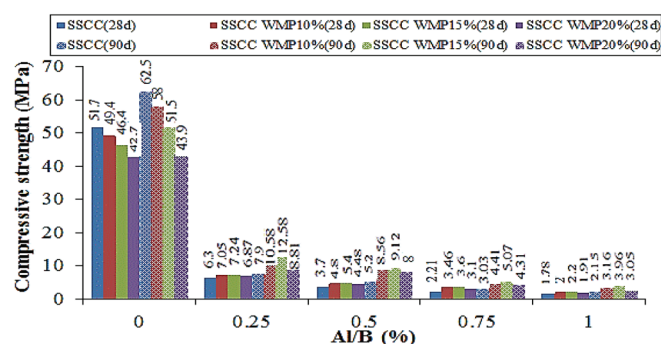
5(c) Waste marble sand cellular concrete (WMSCCs)

Fig. 5 Evolution of CC's bulk density as a function of the aluminum content for different percentages of WMP

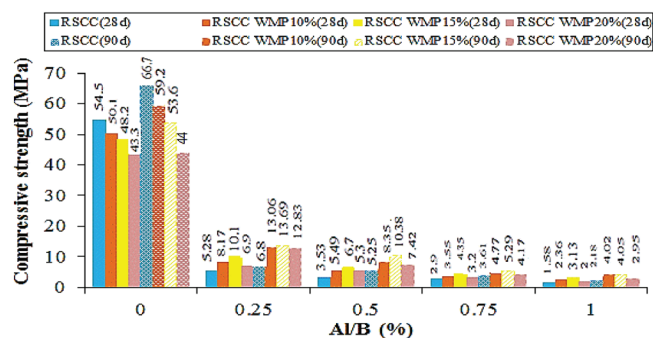
ing ages of 28 and 90 days. Each compressive strength value given in this paper is the average of all three specimens. These findings reveal that the compressive strength decreased noticeably in the mixtures as a function of the aluminium content ranging from 0 to 1%. This is mainly due to the appearance of porosity of the materials developed, since higher amounts of the foaming agent used generally create voids in

the matrix and result in a lower density (Nambiar and Ramamurthy, 2006 a,b). The decrease in compressive strength is more significant for the control samples (reference mixtures) for the zero WMP content. Concerning the effect of the WMP content on the compressive strength, the highest compressive strength was obtained for the SSCC WMP15%, the RSCC WMP 15% (85%C+15%WMP) and for the WMSCCWMP 10%(90%C+10% WMP); the compressive strength results of these compositions was about 5.4MPa, 6.7 MPa, and 4.6 MPa respectively at 28 days and reached 9.12 MPa, 10.38 MPa, and 8.74 MPa at 90 days for the 0.5% aluminum content. According to (Shirule et al., 2012), the pore-filling effect of the fine marble dust enhances the cement matrix and transition zone (TZ) surrounding the aggregate qualities, thereby increasing the cement's compressive strength as the replacement ratio of marble dust rises to 15.0%. Additionally, the waste marble enhances the binding capacity of the concrete matrix at the chemical level. Even though marble is not a pozzolanic material, the reaction between the calcite in the marble powder with the C_3A of cement to create calcium carbo-aluminate results in a compact structure and improves the compressive strength of the mix containing marble powder (Munir et al., 2017; Vardhan et al., 2019). The compressive strength results of the CCs developed for all the WMP contents can be classified as a lightweight concrete for construction and insulation according to the standards required by (RILEM, 1978) other than the compositions above a 0.5% aluminum content, the compressive strength values of which are negligible and are not in accordance with the standards in terms of the classification for lightweight concrete (Ferhat et al., 2023). In contrast, the reduction in compressive strength results was observed at a level of 20% WMP for the SSCC and RSCC, respectively and in a range [15%–20%] of WMP for WMSCC. It is easy to conclude that the microstructure of WMP can affect the strength of the samples and that a higher WMP content is unfavourable for the development of compressive strength due to the decrease in the chemical compounds (C_3S and C_2S) in the cementitious material, which impacts the strength of the concrete (Khodabakhshian et al., 2018) and the inability of the substance to take part in the hydration process.

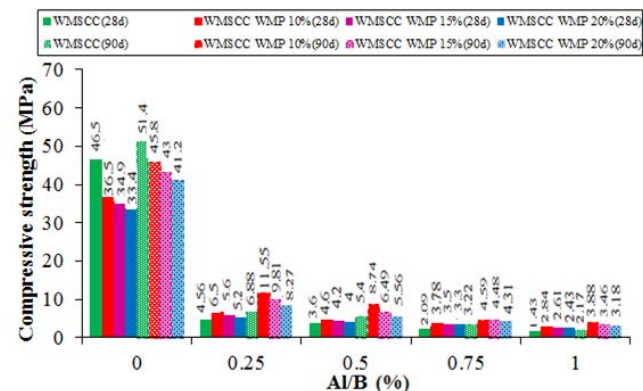
On the other hand, the types of sand differ in their mineralogical nature, grain shape, angularity, proportion of fine elements (Belhadj et al., 2014; Makhoulfi et al., 2014), porosity (Tebbal & Rahmouni, 2016), and particle size (Nambiar and Ramamurthy, 2006 a,b), which negatively affect the increase and decrease in the mechanical properties of the cured cellular concrete because its addition affects the pore size in the paste. The results obtained show that the river sand results in the highest compressive strength, followed by the sea sand and waste marble sand, when the percentage of aluminum varies from 0 to 1%. It is then evident that natural sands with a well-graded particle size distribution and rounded particles exhibit the highest strength compared to waste marble sand with angular particles and a higher proportion of fines. Similar behaviors have been observed, depending on the lifespan (90 days) and compressive strength increases in all the cellular concrete compositions. Figure 7 shows the relationship between the bulk density and compressive strength at 28 days of cellular concretes. A strong relationship was observed between



6 (a) Sea sand cellular concrete (SSCCs)



6 (b) River sand cellular concrete (RSCCs)



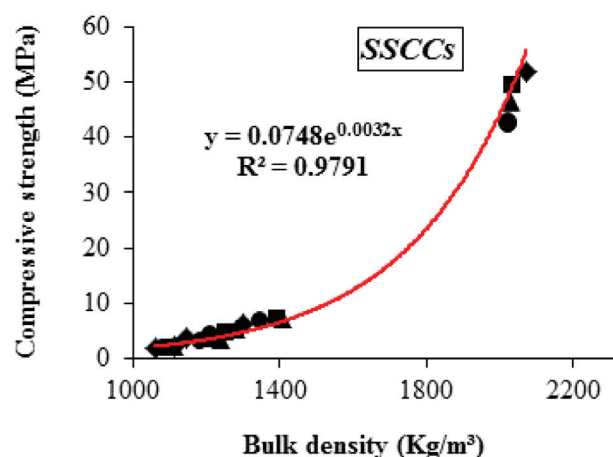
6 (c) Waste marble sand cellular concrete (WMSCCs)

Fig. 6 Evolution of CC's compressive strength as a function of the aluminum content for different percentages of WMP

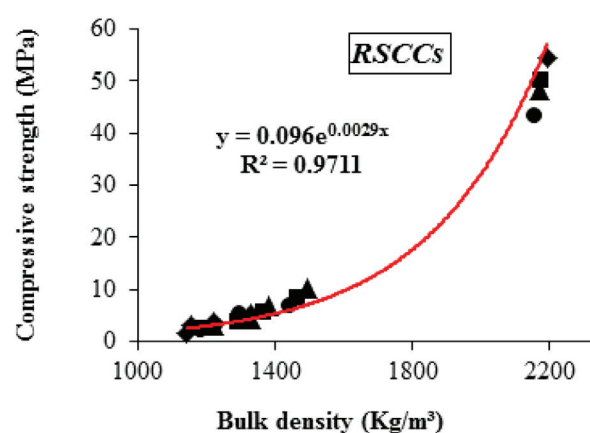
the bulk density and compressive strength, with a coefficient of correlation of 0.979, 0.971, and 0.988 for SSCCs, RSCCs, and WMSCCs, respectively.

3.3 Thermal conductivity

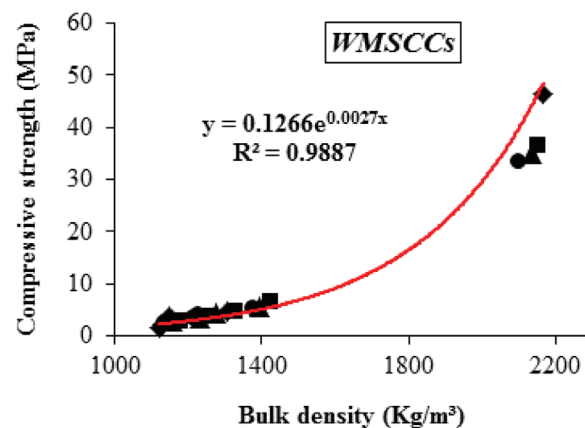
Figure 8 describes the evolution of the thermal conductivity of the cellular concrete developed as a function of the aluminum content ranging from 0 to 1% for the various WMP percentages. The results show that the thermal conductivity decreases as a function of the aluminum content (Damene et al., 2014). However, the thermal conductivity decreases significantly in compositions free of WMP (the reference mixture) compared to those containing WMP. It can be seen that the proportion of higher porosity in the matrix and the distribution of voids in the material is a key factor in the material's lightness and reduced thermal conductivity (Lian et al., 2011). Regarding the effect of the WMP content, the variations of these parameter values is very similar



7(a) Sea sand cellular concrete (SSCCs)



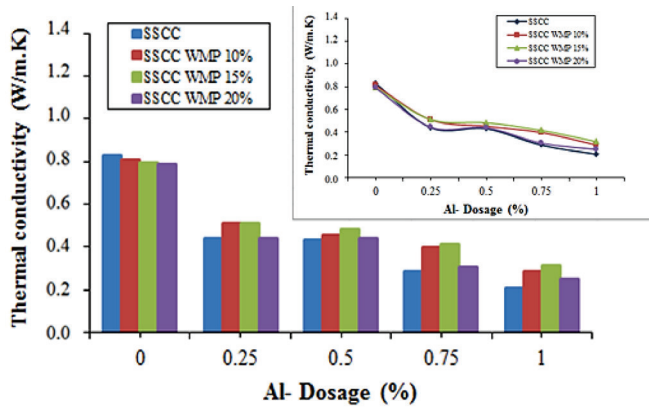
7(b) River sand cellular concrete (RSCCs)



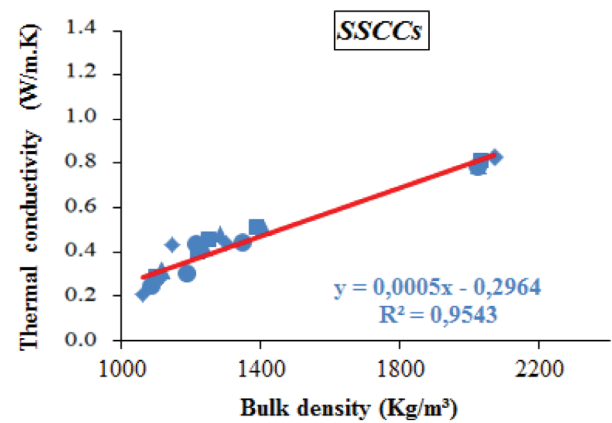
7(c) Waste marble sand cellular concrete (WMSCCs)

Fig. 7 Bulk density vs. compressive strength relationship of CC's

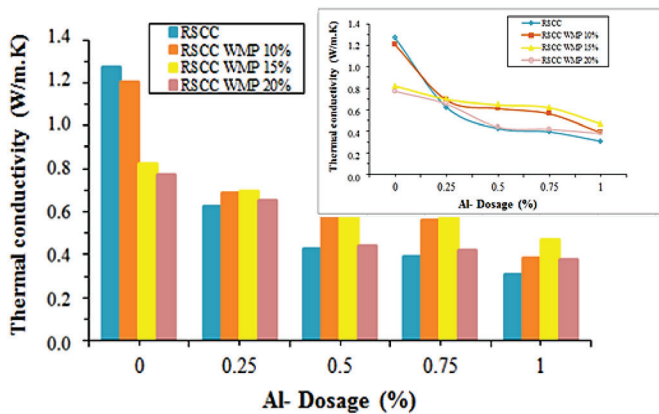
to the density, i.e., a lower density equals better conductivity. The ratio of the decrease was about 75% for the SSCC without WMP to 60% for the SSCC WMP 15% (Fig. 8(a)), and reached 76 % for the RSCC to 43 % for the RSCC WMP 15% (Fig. 8(b)), and on the order of 67.5 % for the WMSCC to 57 % for the WMSCC WMP 10% (Fig. 8(c)) when the aluminum dosage varied from 0 to 1%. The results of the thermal conductivity values obtained for all the compositions created from the various types of sand are in accordance with the recommendations (RILEM, 1970) for lightweight concrete for construction and insulation,



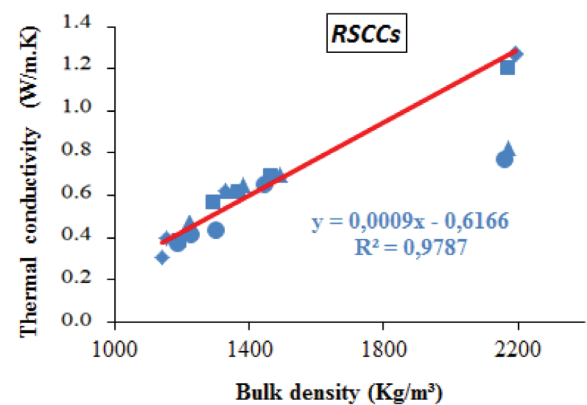
8 (a) Sea sand cellular concrete (SSCCs)



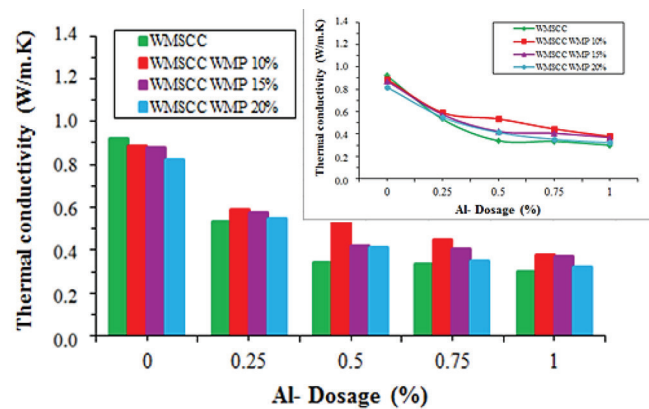
9 (a) Sea sand cellular concrete (SSCCs)



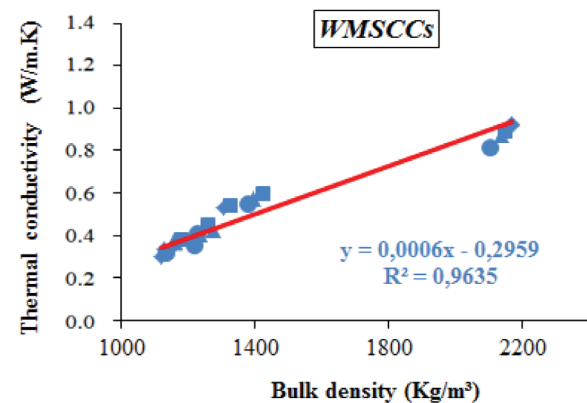
8(b) River sand cellular concrete (RSCCs)



9 (b) River sand cellular concrete (RSCCs)



8(c) Waste marble sand cellular concrete (WMSCCs)



9 (c) Waste marble sand cellular concrete (WMSCCs)

Fig. 8 Evolution of CCs' thermal conductivity as a function of the aluminum content for different percentages of WMP

which specify thermal conductivity coefficients of less than 0.75 W/(m.k). The sea sand and waste marble sand yield the lowest thermal conductivity, followed by the river sand, suggesting that smaller sand particle sizes have reduced thermal conductivity and proving that finer sands are better at improving the thermal insulation properties compared to coarser sand particles that produce a higher values of thermal conductivity.

Since the thermal conductivity of cellular concrete is significantly influenced by its bulk density. Figure 9 illustrates the strong relationship between these two variables with a coefficient of correlation of 0.954, 0.978, and 0.963 for SSCCs, RSCCs, and WMSCCs, respectively.

Fig. 9 Thermal conductivity vs. bulk density relationship of CC's

4 CONCLUSIONS

This research investigated the effect of the type of sand, marble waste powder, and aluminum dosage on the physico-mechanical and thermal properties of cellular concrete. The main findings are as follows:

- In the case of cellular concrete, with the addition of WMP, the bulk density results increase to a level [0–15%] of WMP content from the mixture containing sea and river

sand (SS and RS) and to a level [0–10%] of WMP for the mixture formed with waste marble sand (WMS). Concerning the effect of the aluminum content, the density decreases with increasing the dosage of aluminum.

- The concrete's compressive strength decreases significantly as a function of the aluminum content. The experimental findings show that the compressive strength increases up to a WMP content of 15% before starting to decline at a WMP content of 20% for CCs made with RS and SS. Additionally, the compressive strength of CC compositions containing WMS decreases by the incorporation of WMP content higher than 10%. In general, 10%–15% WMP is the perfect replacement level for cement in terms of the mechanical properties of cellular concrete under the given curing conditions.
- The thermal conductivity of the CCs developed is related to their density and decreases as the aluminum dosage increases as a result of the porosity in the matrix.
- Strong correlations between bulk density vs. compressive strength, and thermal conductivity vs. bulk density of CCs were established.

- By combining the physico-mechanical and thermal characteristics as shown in this study, it can be concluded that it is possible to produce quality cellular concrete with 10-15% WMP as a partial replacement for cement. The selection of the type of sand is important in achieving the desired properties of cellular concrete in accordance with the standards required by RILEM for the classification of lightweight concrete, even though the river and sea sand offer better qualities, waste marble sand has the potential to be an alternative in the production of cellular concrete.

Acknowledgements

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EFFECT OF RECOVERED FIBERS ON THE BEHAVIOR OF CONCRETE IN TENSION

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Abstract

This study evaluates the performance of concrete under the effect of the addition of non-straight fibers recovered during the machining of steel parts and then studies the effect of the addition of corrugated fibers (chips) on their behavior in direct traction of the fiber concrete. In order to characterize the mechanical behavior and tearing of these fibers, three lengths of fibers were used (4 cm, 5 cm, and 6 cm); for each length we varied the percentage by volume (0.3%, 0.5%, 0.8%, 1%, and 1.2%). To make a comparative study, a second series of tests was carried out with control concrete (BT) without any fibers of the same composition. The results of the tests carried out indicate that the resistance and stiffness are significantly improved and that the fibers imparted significant ductility to the material compared to the control concrete. It was also observed that the percentage of fibers significantly influences the appearance of cracks. Moreover, with an increase in the percentage of fibers from 0.3% to 0.8%, the resistance increases, but beyond a content of 0.8%, the mechanical characteristics decrease.

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Key words

- Concrete,
- Chipped fibers,
- Characterization,
- Direct tensile,
- Ductility,
- Breaking.

List of Symbols

E: Modulus of elasticity of the concrete in compression
 e_p : Thickness of fiber “chips”
L: Fiber length
 L_r : Cps fiber width
n: Undulations (spirals)
v: Percentage by volume of the fibers,
 ε_c : Strain at σ_c
 ε_{cp} : Strain at σ_{cp}
 σ_c : Crack initiation stress
 σ_{res} : Residual stress
 σ_{cp} : Maximum post-cracking stress
 γ : Fiber Density
 σ_e : Tensile strength of fibers

1 INTRODUCTION

Steel fibers are increasingly used for the reinforcement of concrete, especially in the case of thin elements, which can replace or supplement conventional reinforcement, depending on the type of structure. In the case of slabs (thin elements) and shotcrete (vaults), a gain in productivity can be observed. As the concrete matrix has a brittle nature and a low tensile strength, the mechanical behavior in tension is conditioned by the cracking of the concrete. The incorporation of the fibers in concrete makes it possible to mitigate the behavior of structures in the face of cracking (Tadepalli et al., 2015; Gali et al., 2017; Das et al., 2020); thus the composite will have better tensile strength.

The main role of fibers is:

- On the scale of the material by “stitching” the micro cracks, the fibers delay the appearance of cracks. This action can cause an increase in tensile strength, which depends on the percentage of fibers incorporated into the concrete and their orientation in the concrete mass.
- On the scale of the structure, by “retaining” the cracked concrete, thanks to their anchoring on both sides of the crack. The fibers make it possible to attenuate the fragility of the concrete and thus provide it with greater ductility.

Fiber-reinforced concrete has been the subject of a great deal of research around the world for the last forty years; various studies have been conducted to characterize the behavior of this type of composite and to understand the behavior of the elementary fiber-matrix association.

According to his experimental study (controlled tests), Zhan (1991) showed that the presence of fibers allows for an increase of approximately 16% in the tensile strength of a material. The ductility of the post-rupture behavior of the composite is improved (Bouafia et al., 2002; Rossi et al., 1989). Chanvillard (1993) carried out tests on the pull-out of an individual fiber by an experimental program, including the effect of the length of a corrugated fiber, its orientation, and the matrix. The author was able to highlight the phenomena that govern the modes of rupture during the pull-out test. Casanova and Rossi (1996) conducted a uniaxial tensile test to obtain an intrinsic crack stress-opening relationship. According to the authors, the calculation of the equilibrium of the forces in a cracked section makes it possible to know the resistance capacity in flexion. An experimental study of the mechanical behavior of reinforced concrete with chips was made by Djebali et al. (2011) using direct tensile tests. The authors concluded that the fibers substantially improved the stiffness and the strength of the concrete. Zia et al. (2023) conducted a study to explore and evaluate the efficiency of raw steel fibers from waste tires (RSFs), which were locally available in Bratislava, Slovakia, for improving the durability, and strength properties of local concrete. Based on the outcomes of this study, the raw steel fibers from discarded tires have the potential to be used for producing sustainable and durable concrete if used in pure and proper shapes. In another study carried out by Zia et al. (2023), the objective was to investigate the influence of the combined effect of discarded tire steel fibers and industrial steel fibers on the properties of fiber-reinforced concrete. The authors noticed significant improvements in the properties of the hybrid fiber-reinforced concrete compared to zero-fiber concrete (Zia et al., 2023 a, b).

Several other authors have highlighted this phenomenon during their experimental studies. The studies carried out on this subject reveal certain contradictions as to the objectives aimed at by fiber reinforcement. Markovic et al. (2003) argued that the main purpose of adding fibers to concrete is to increase its tensile strength and ductility, whereas for (Kawamata et al., 2003), the fibers are not introduced for the purpose of improving the concrete's tensile strength, but mainly to control cracking and obtain quasi-ductile behavior after cracking by bridging the cracks.

Under tensile stress, cracks develop in the matrix, and the fibers can sew them together, thus increasing the resistance to the opening of cracks and delaying their propagation.

On the other hand, other researchers have carried out experimental studies in order to replace reinforced concrete with fibers likely to provide the concrete with good resistance to traction, bending, and shear (Atlaoui and Bouafia, 2008; Wu et al., 2003; Noghabai, 2000; Soroushian et al., 1992; Mobasher and Shah, 1990; Abdallah et al., 2017; Abdallah et al., 2016b; Ding et al., 2011; Deng et al., 2018; Isla et al., 2015; Li and Liu, 2016; Barragán, 2004; Johnston, 1996; Bouafia and Adjrad, 1997; Lim, 1987). At first, researchers tried to increase the compressive strength and flexural strength of concrete by adding fibers, but the results obtained were limited (Sukontasukkul, 2004). It has been observed that the main role of fibers in a cementitious material can be assessed under two aspects:

- The control of the propagation of a crack in a material in a state of service by reducing the opening of the cracks;
- The transformation of the brittle behavior of a material into a ductile behavior, which increases safety during ultimate loading states.

Embedded steel fibers are dispersed in concrete in order to increase the tensile behavior and its resistance to impacts, as well as to achieve a better degree of ductility (Abdallah et al., 2018 a, b; Abdallah et al., 2018b; Adjrad et al., 2016; Tadeballi et al., 2013). Among these aims, the greatest benefit of including steel fibers in concrete is to achieve a quasi-ductile behavior after cracking in an otherwise brittle material. The steel fibers therefore allow the concrete to continue to support loads even after cracking; this phase is called “post-cracking behavior” (Abdallah et al., 2016b; Tadeballi et al., 2013). Another benefit of adding steel fibers when evenly distributed throughout a concrete matrix is to achieve very effective crack control as there will always be fibers close to a micro-crack, thereby preventing the crack from spreading. Other than steel fibers, the incorporation of polypropylene fibers in concrete improves the long-term durability of hydraulic structures by reducing the number of voids and shrinkage cracks (Zia and Ali, 2018). In an attempt to answer all these questions, we have chosen to focus our research work on the study of the effect of non-straight fibers recovered from industrial waste (chips) on the behavior of concrete in direct tension. The direct tensile test is the most suitable one for characterizing the behavior of a material subjected to a longitudinal action.

The main objective of this study is to characterize the behavior of metal fiber recovered from machining waste added to concrete; for this purpose, experimental direct traction and compression tests on specimens were carried out. To make a comparative study, a second series of tests was carried out with the same fiber-free concrete.

This experimental study has been carried out on standard cylindrical specimens for the compression tests and on specimens in the shape of dumbbells with square sections for the direct tensile tests, which required the design and realization of a special device for fixing the specimens on a traction machine. Various fiber contents (0.3%, 0.5%, 0.8%, 1%, and 1.2%) were incorporated into a concrete matrix with a sand/gravel (S/G) ratio equal to 0.8; all of the results were then exploited qualitatively from the experimental physical observations.

2 CHARACTERISTICS OF THE FIBERS USED IN THE STUDY

The steel fibers used in the study are local fibers resulting from the machining waste of steel parts, i.e., “chips”, which were recovered at the level of the National Company of Industrial Vehicles in Algeria (SNVI). They have a rather undulating geometric shape, which favors anchoring them in concrete; the shape of these fibers is shown in Fig. 1. The mechanical and geometrical properties of the fibers used are given in Tab. 1: three lengths of fibers were used (4 cm, 5 cm, and 6 cm) with 3, 5 and 6 undulations (spirals) for each length; the various percentages as to the volume tested are given in Tab. 2.

The experimental program consisted of carrying out direct traction tests with controlled deformations. Three tests for each type of combination were carried out; the anchoring of the ends of the fibers in the clamping jaws of the hydraulic press was improved by fiberglass resin (see Fig. 2).



Fig. 1 View of the fibers used in the study



Fig. 2 Anchoring system

Tab. 1 Characteristics of the mechanical properties of the fibers

Fibre type	L _f (cm)	L _r (cm)	e _p (cm)	γ (kg / m ³)	σ _e (MPa)
Chips	4- 5-6	0.2	0.06	5818	280

Tab. 2 Percentage of fibers incorporated

Weight (kg)	17.45	29.09	46.54	58.18	69.8
% in volume	0.3	0.5	0.8	1.0	1.2

3 PROCEDURE FOR THE PREPARATION OF THE TEST SPECIMENS AND THE MIXING PROCEDURE

It was decided that only one concrete composition would be used; the specimens were designed and produced by keeping the amounts of the ingredients exactly the same using ordinary Portland cement of the type CPJ-CEMII/B 42.5 R NA 442 of the class 42.5 (Lafarge cement region of M'sila (LCM), Algeria); the aggregates used (a crushed shape) come from the Tizi-ouzou region (quarry rocks). The composition of the concrete for 1m³ was determined by the Dreux-Goris experimental method (Dreux and Festa, 1998). Tab. 3 summarizes the proportions of all the mixed batches employed in this testing program. The granulometric curves for each type of aggregate are presented in Fig. 3. The concrete was mixed in a rotary mixer, while respecting the following scrupulous procedure:

- Introduction of sand, the aggregate and cement, mixed for 60 seconds,
- Introduction of the water, mixed for 40 seconds,
- Introduction of the superplasticizer, mixed for 30 seconds,
- The fiber concrete mixes were introduced last in small quantities; with each addition, there was mixing for 20 seconds.

Tab. 3 Proportions of the concrete mixture used

Ingredient	Amount (kg/m ³)
Coarse aggregate (8/15)	895
Medium aggregate (3/8)	171
Sand (0/3)	753
Portland cement CPJ CEMII/A 42.5 (c)	380
Water	207
Superplasticizer (0.5% of cement weight (ml))	190

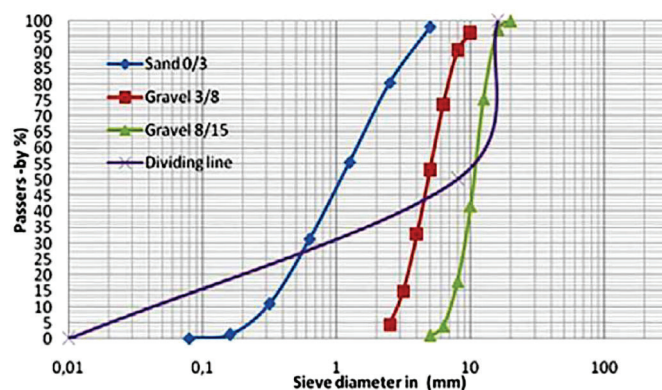


Fig. 3 Grading curve

4 EXPERIMENTAL PROCEDURE

4.1 Fiber characterization tests

The tests were carried out on an “Ibertest” hydraulic press with controlled deformations at the Materials and Structures Laboratory of Civil Engineering at the Mouloud Mammeri University of Tizi Ouzou in Algeria. The press was equipped with a digital control command (see Fig. 4). The geometric charac-



Fig. 4 Tensile test device

teristics were introduced automatically, and the useful length of the fiber was 100 mm. The loading speed was 20 mm/min.

4.2 Compression tests

The compression tests were carried out on an AUTOTEST brand hydraulic press with a capacity of 2000 KN (Fig. 5) on a cylinder with a 16 cm diameter and 32 cm height (according to the NF EN12390-4 standard) as illustrated in Fig. 6).



Fig. 5 Hydraulic press



Fig. 6 Cylindrical specimen

4.3 Geometric specimen

The specimens used were dumbbell specimens with a cross section of $[100 \times 100]$ mm² (Fig. 7). They were provided with a U-shaped notch (5 mm deep by 5 mm opening) with a slight undercut on the sides to facilitate stripping of the specimen. Fig. 8 shows the formwork of the tensile specimens.

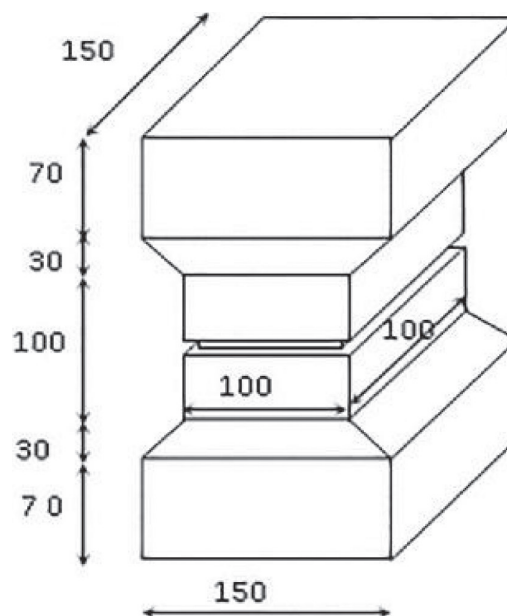


Fig. 7 Fiber concrete specimen



Fig. 8 Formwork for tensile specimens

4.4 Specimen clamping device

To fix the specimens (Fig. 9) on the jaws of the tensile machine, a special device was designed with the help of the value analysis method (Fig. 10).

The device was adapted to the “IBERTEST” 200KN traction machine. It consists of two identical parts, which are mounted on the upper and lower jaws of the traction machine using the fixed jaws. The specimens are wedged inside the device on the corner supports. The adjustment screw makes it possible to adjust the position of the sliding jaws in order to be able to receive specimens of different sizes. The coincidence of the axis of the specimen with the axis of the machine is ensured by stops. Because the device is integral with the jaws of the traction machine, it is not necessary to provide it with a ball joint. The vertical force was applied gradually at a controlled loading speed (i.e., 0.005 MPa/second). The Wintest32 software programmed for this press records the value of the vertical force and the corresponding deformation for each step, as well as the stress as a function of the deformation for each loading.



Fig. 9 Tensile test device

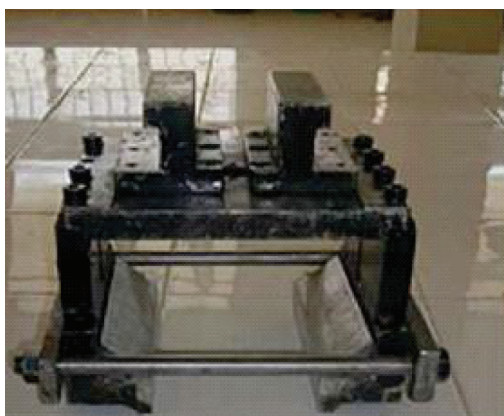


Fig. 10 View of specimen clamping device

5 TEST RESULTS AND ANALYSIS

5.1 Tensile fiber characterization tests

The tension stress-strain curves for the three lengths of fibers (4, 5, and 6 cm) containing six undulations along their length are demonstrated in Fig. 11. During the test, it was ob-

served that the undulations of the fiber gradually open up until the fiber flattens. Beyond that, a ductile fracture of the steel can be observed. The tensile strength increased as a function of the number of undulations; it reached $R_m = 280$ MPa for a length $L = 6$ cm with 6 undulations shown in Fig. 12.

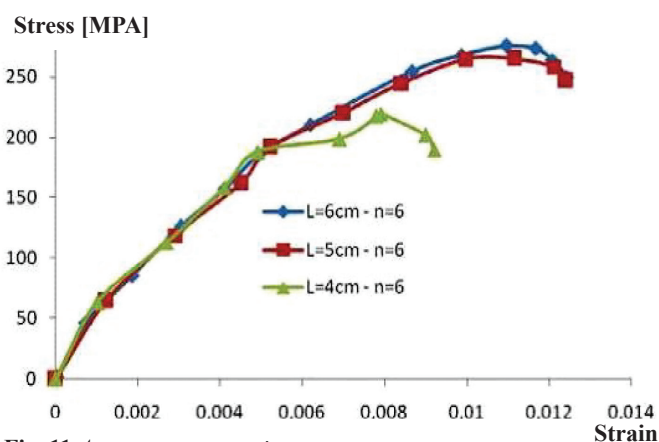


Fig. 11 Average stress-strain curve



Fig. 12 Fibers used: $L = 6$ cm with 6 undulations

5.2 Compression test results

The stress-strain compression curves of the concrete specimens are given in Fig. 13.

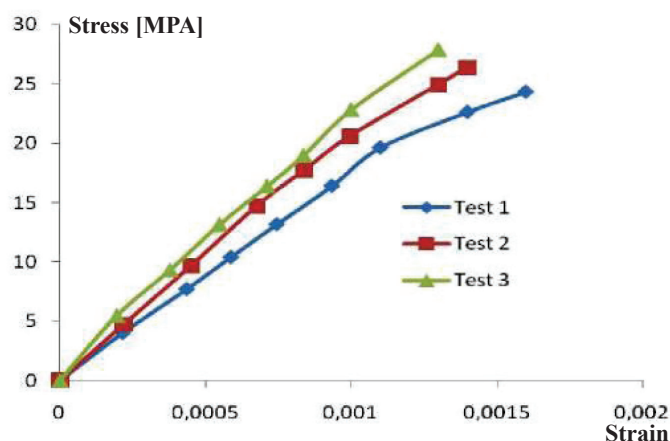


Fig. 13 Stress-strain curves in compression

The experimental mechanical properties of the concrete obtained after 28 days of curing are presented in Tab. 4.

Tab. 4 Mechanical characteristics of the concrete

Tests	Compressive strength [MPa]	Elastic modulus [MPa]
Test 1	24.30	30934.00
Test 2	26.39	29305.00
Test 3	27.88	33932.00
Average	26.19	31390.33

Fig. 13 shows the superposition of the average stress-strain curves of the three series of tests. Tab. 4 presents the mechanical characteristics of the concrete. Based on the outcomes and values examined, the average compressive stress determined from the composition concrete is 26.19 MPa; according to the European standard, it is a C25/30 grade concrete, which is recommended for the construction of the load-bearing elements of buildings. The elastic modulus is 31390.33 MPa, which corresponds to a standard C25/30 class concrete (Eurocode 2, 1992).

5.3 Characterization of the tensile strength of the fiber-reinforced concrete by direct tests.

The results obtained by the direct traction tests on the prismatic specimens of the fiber-reinforced concrete with different fiber contents ($V_f=0\%$, $V_f=0.3\%$, $V_f=0.5\%$, $V_f=0.8\%$, $V_f=1\%$, and $V_f=1.2\%$) are shown in Fig 14.

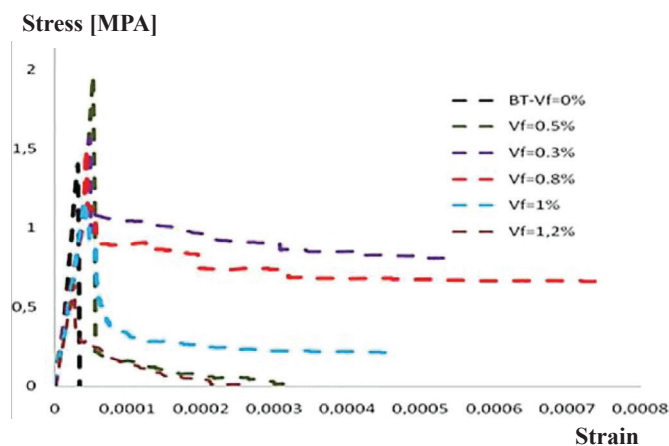


Fig. 14 Direct tensile stress-strain curves

Fig. 14 indicates that the fibers improve the rigidity and the resistance of the composite for the contents ($W=0.3\%$, $W=0.5\%$, and $W=0.8\%$); on the other hand, for the contents ($W=1\%$ and $W=1.2\%$), the strength decreases relative to the control concrete, which is explained by the fact that the percentage is very high, which creates a lot of voids in the cementitious matrix to form the corrugated fiber spirals.

The results obtained showed that the addition of fibers makes the ductility of the material significant compared to the control concrete (without fibers); on the other hand, the behavior of the fiber-reinforced concrete in tension with contents (0.3%, 0.5%, 0.8%, 1%, and 1.2%) are characterized by the presence of two phases. A linear phase corresponding to the quasi-elastic behavior of the material is the pre-cracking of the concrete. This phase ends with the appearance of a macro crack. A second phase in which a drop in stress was observed due to the cracking of the concrete matrix; in this phase the fibers intervene by bridging the cracks and limiting their opening, which gives the material a significant deformation potential, and therefore ductile behavior in traction.

5.4 Analysis of the direct tensile strength results and failure modes

The results obtained (Tab. 5) showed that the addition of fibers improves the rigidity and the resistance of the composite and gives a significant amount of ductility to the material compared to the concrete without fibers ($W=0\%$). This has also been demonstrated by (Bouafia et al., 2002). The increase in the resistance of the composite reached 41% of the control concrete (the concrete with 0.5% fibers), while the increase in the modulus of elasticity is increased by 13% (for $W=0.5\%$ of fibers), whereas for the fiber contents $W=0.3\%$ and $W=0.8\%$, they are of the order of 11.42% and 7.14% respectively, compared to the control concrete. The shape of the traction curves obtained is identical to that given by Lim (1987). On the other hand, the tensile behavior of the fiber concrete is characterized by the presence of two phases: (1) a linear phase corresponding to the quasi-elastic behavior of the material; in this phase cracks appear; (2) a second phase in which we can observe a sudden drop in stress following the rupture of the concrete matrix, which has reached its cracking stress. At this level, the composite is stressed in tension; the fibers intervene by bridging the cracks and limiting their opening. They provide the material with its residual bearing capacity and thus make it possible to avoid a sudden break.

On the other hand, it appears that the post-cracking stress (σ_{cp}), the residual stress (σ_{res}), and the modulus of elasticity increase according to the percentage by volume of the incorporated fibers up to 0.5%, but beyond this percentage, these parameters decrease. The decrease in the concrete's strength and residual stress value (σ_{res}), with fiber contents higher than 0.5% could be explained by the fact that a high fiber content would create porosity and cavities in the concrete due to the spiral shape of the chips. Indeed, the fibers are supposed to be distributed in a uniform way, but this is not always the case because a multitude of fibers stick together during the integration of the fibers and the mixing.

The most advantageous fiber content from a strength point of view is 0.5%. Beyond 1%, the fibers are of no significant interest apart from providing ductility. For a content of 1.2%, the strength of the fiber-reinforced concrete decreases, and the implementation of the mixture become difficult. The optimum percentage of the fibers can be set at 1%.

Tab. 5 Measured mechanical characteristics

V_f (%)	σ_c [MPa]	$\epsilon_c \times 10^5$	E [MPa]	σ_{cp} [MPa]	$\epsilon_{cp} \times 10^{-5}$	σ_{res} [MPa]
W= 0% (BT)	0.41	1.19	31,390.33	1.40	3.08	0.00
W= 0.3%	0.40	1.31	30,092.00	1.56	4.76	1.06
W= 0.5%	0.42	1.30	33,089.00	1.97	5.20	0.23
W= 0.8%	0.27	0.91	29,406.00	1.50	4.27	0.90
W= 1%	0.30	1.03	29,879.00	1.18	4.06	0.31
W= 1.2%	0.21	0.82	26,281.00	0.65	2.52	0.28

6 CONCLUSIONS

Within the framework of this experimental study, the mechanical device designed was put to a test with success; it made it possible to carry out direct tensile tests on concrete test specimens of fibers from waste parts machined in steel, despite the complexity of this type of test. The recorded stress-strain curves allowed us to characterize the behavior of this type of material compared to concrete alone (without fibers).

This study allowed us to highlight the effect of the length of the fibers and the number of undulations on their tensile strength. It appears that the resistance is achieved for a length of $l = 6 \text{ cm}$ $n = 6$ undulations. In the tensile test, the corrugations tend to flatten out before the steel fiber elongates.

Adding chips to a concrete matrix improves the tensile strength and stiffness of the composite and gives it significant ductility. This has also been demonstrated by (Bouafia et al., 2002). This improvement is significant for a fiber content of $W = 0.5\%$. The increase in the strength of the composite is approximately 41% greater than that of the control concrete (without fibers), while the increase in the modulus of elasticity is increased by 13% (for $W=0.5\%$ of fibers).

On the other hand, the bending behavior of fiber concrete is characterized by the presence of two (2) phases: A linear stage, corresponding to the quasi-elastic behavior of the material; it is the pre-cracking phase of the concrete. This phase ends with the macro-cracking. Then there is a second phase in which there is a sudden drop in the bearing capacity of the material (without a sudden rupture of the concrete section); this is the post-cracking phase, which stabilizes at the time of the tensioning fibers, thus providing post-cracking ductility. In this phase, the concrete matrix breaks, and the edges of the cracks are connected by the fibers, thereby avoiding a sharp break.

On the other hand, it appears that the post-cracking stress (σ_{cp}), the residual stress (σ_{res}), and the modulus of elasticity increase according to the percentage by volume of the fibers incorporated up to 0.5%, but beyond this percentage, these parameters decrease. The drop in the strength of the concrete and in the value of the residual stress for the concrete whose content is higher than 0.5% of the fibers could be explained by the fact that a high fiber content could introduce a large number of defects (cavities), due to the spiral shape of the chips.

Indeed, the core of the rolls of the chips is not always filled with concrete; when it is, it is only filled with cement paste or fine sand. It should also be noted that the introduction of chips into the concrete prevents spalling and a sudden rupture of the concrete. The most advantageous fiber content from a strength point of view is 0.5%. Beyond 1%, the fibers are of no significant interest except for their contribution of ductility. The saturation threshold can be set at $W=1.2\%$. For this value, we have a drop in resistance and a difficulty in the implementation of the concrete.

Finally, we can say in view of the results obtained, the machining chips can be upgraded by their use in the preparation of fiber concrete intended for paving concrete pavements and repairs of road surfaces. This concrete can also be used to increase the fire resistance of reinforced concrete because the fibers could limit the opening of cracks and thus protect the traditional reinforcements from thermal radiation.

It could be interesting in the future to carry out tests on actual specimens of large dimensions and to carry out tests with flat fibers instead of wavy ones in order to reduce the voids (cavities) in the cement paste and also to associate these fibers with traditional reinforcements.

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ANALYSIS OF THE INDOOR CLIMATE IN A SMALL, NEWLY-BUILT AUDITORIUM

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Abstract

The article focuses on an analysis of the indoor climate in a small, newly-built auditorium. Providing optimal indoor climate parameters in the interiors of a university is extremely important for students. Meeting these parameters is essential not only from a physiological point of view, but also to achieve desirable student performances. The parameters of CO₂ concentration are also influenced by the air distribution system in a small auditorium. The correct positions of the supply and extract air is very important. The first experimental measurements of CO₂ concentration were carried out in a small auditorium in an old building. The natural ventilation of the small auditorium was evaluated. The second experimental measurements of the CO₂ concentration were made in a small auditorium in a newly-built building. The ventilation system and the modern air distribution system of the small, newly-built auditorium were evaluated based on the CO₂ concentration and the PMV index. A Testo 480 was used for the measurements. The conclusion of this paper contains the recommendations for the design of a newly-built ventilation system in a small auditorium.

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Key words

- CO₂ concentration,
- Small auditoriums,
- Ventilation system.

1 INTRODUCTION

Sophisticated ventilation systems in auditoriums are symptomatic of newly-built university buildings. The research conducted on the CO₂ concentration in a small auditorium in Slovakia pointed out a very serious problem represented by a lack of forced ventilation. Natural ventilation through windows during breaks is insufficient. Therefore, it is necessary to reconstruct existing small auditoriums from the point of view of forced ventilation to complete a mechanical ventilation system.

Today, small auditoriums cannot work without a forced air exchange (Etheridge, 2011). Therefore, when planning new auditoriums, it is necessary to opt for a high-performance ventilation system (Heiselberg, 2006), (Awbi, 2003). The architectural design of small auditoriums in the first stage of an

architectural study must be done in such a way that it allows for the installation of a high-performance ventilation system (Santamouris and Wouters, 2003), (Krajčák et al., 2000). Existing small auditoriums need to be renovated in terms of their ventilation; forced ventilation needs to be installed. Of course, this requires minor architectural modifications.

It must be ensured that the permissible concentration level of CO₂ for students in a small auditorium is a priority (Bánhidi and Kajtár 2000). A high CO₂ level causes problems with concentration and results in a feeling of fatigue, respiratory diseases, the development of allergies, etc. (Hall and Hall 2020). Students spend the majority of their time in auditoriums; therefore, ensuring high-quality ventilation is essential nowadays. A high-quality ventilation system is a basic prerequisite (STN

EN ISO 7730, 2006). Choosing the right air distribution system is another requirement, because a draft caused by an incorrect position (STN EN ISO 7726, 2003), (Krajčík et al., 2021), the distance of the air supplied and air extracted from the floor, and the wrong air flow speed can significantly affect the thermal comfort in a small auditorium. In today's energy crisis, ensuring the parameters of an indoor climate must also be optimised from an energy point of view (Földváry et al., 2015). The research was focused on evaluating the CO₂ concentration levels, the PMV index (Predicted Mean Vote) and the operating temperature in order to test the quality of the indoor climate of two small auditoriums.

2 RESEARCH ON AN INDOOR CLIMATE IN A SMALL AUDITORIUM IN AN OLD BUILDING

Experimental measurements were carried out in a small auditorium in an old building without a forced ventilation system in the winter period in early 2023. The auditorium at the Faculty of Architecture and Design, Slovak University of Technology in Bratislava, was chosen for the measurements. Lectures are held in that auditorium. The aim of the measurements was to record the parameters, the CO₂ concentration, and the operating temperature at a height of 1.1 m above the floor level in the centre of the auditorium. In the two-hour lecture selected, the number of students was 84. The parameters of the CO₂ concentration were recorded with a Testo 480 device. The outdoor air temperature and the air relative humidity were measured and recorded by the separate device. The outdoor air temperature increased from the value 9.1 °C to the value 9.5 °C. The outdoor air relative humidity decreased from the value 65.3 % to the value 65.5 %. The auditorium has five windows measuring 2.3 x 2.3 m. The ventilation area of the windows was 2.3 x 1.4 m. There is a column radiator under each window.

The results of the measurements were very surprising. Twenty minutes after the start of the lecture, the permissible CO₂ concentration level was exceeded. A value of 1900 ppm was measured. The operating temperature values were 26.5 °C. The attention and concentration of the students dropped significantly. After forty minutes, the CO₂ concentration was so high that a forced thirteen-minute break was necessary during the lecture. A value of 3100 ppm was measured. The operating temperature had an unacceptably high value of 29 °C. The students' heads were drooping down from tiredness, and their attention was minimal. The thirteen-minute break significantly disrupted the lecture flow and shortened the lecture by that amount of time. During the break, five large windows and the door of the auditorium were opened. A large number of students left the auditorium, so it was possible to make a cross draft in the auditorium. After the enforced break, the lecture continued. The values of the CO₂ concentration were permissible, but not optimal; a value of 1200 ppm was measured. The operating temperature dropped to 21 °C, which is a permissible value, but is not optimal. This method of natural ventilation significantly increased the energy consumption.

These experimental measurements showed that natural ventilation for a small auditorium is highly inappropriate.

From the point of view of the students' attention and concentration, this type of ventilation was unsatisfactory, significantly disrupted the lecture, and shortened its duration.

3 RESEARCH ON AN INDOOR CLIMATE IN A SMALL AUDITORIUM IN A NEWLY-BUILT BUILDING

Experimental measurements were carried out in a small auditorium in a newly-built building with a modern air distribution system at the Vienna University of Economics and Business, Fig. 1. The shape of the floor of the small auditorium is sloped. Horizontally continuous vents for the air supply are located in the individual steps of the floor in the small auditorium. The modern shape, position, and location of the vents for the air supply is very progressive for the ventilation system and the architectural solution. The air distribution system is from the bottom upwards. Therefore, the location of the air extraction vents is located up in the ceiling.



Fig. 1 Research on experimental measurements in a small auditorium

3.1 Methodology of the Indoor Climate Research

The aim of the measurements was to record the parameters of the CO₂ concentration and PMV index. The measurements of the CO₂ concentration and PMV index were carried out in the small auditorium at the height of 1.1 m above the floor level at four standpoints, Fig. 2:

- Standpoint A – right side, ninth row of seats, next to the stairs against the inner wall;
- Standpoint B – right side, sixth row of seats, next to the stairs by the inner wall;
- Standpoint C – left side, sixth row of seats, next to the stairs by the outer perimeter wall;
- Standpoint D – left side, ninth row of seats, next to the stairs by the outer perimeter wall.

There were a full number of students at the lecture, so measurements could not be taken in the center of the space. The number of students in this lecture was 59. The total capacity of the auditorium was 116, which means that the auditorium occupancy was 50%.

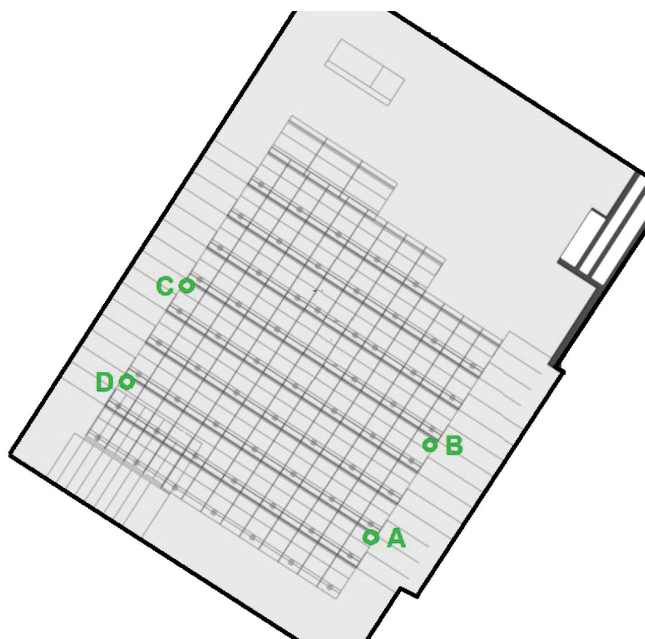


Fig. 2 Positions of the standpoints in the small auditorium

The parameters of the CO_2 concentration and PMV index were recorded with the Testo 480 device, Fig. 3. The input data for PMV index measurements were a metabolic rate of 1.0 met and clothing insulation of 1.0 clo. Twenty measurements were made, one after the other, with a time shift in the individual positions. The values measured were statistically averaged. Measurements were made separately at the individual positions.

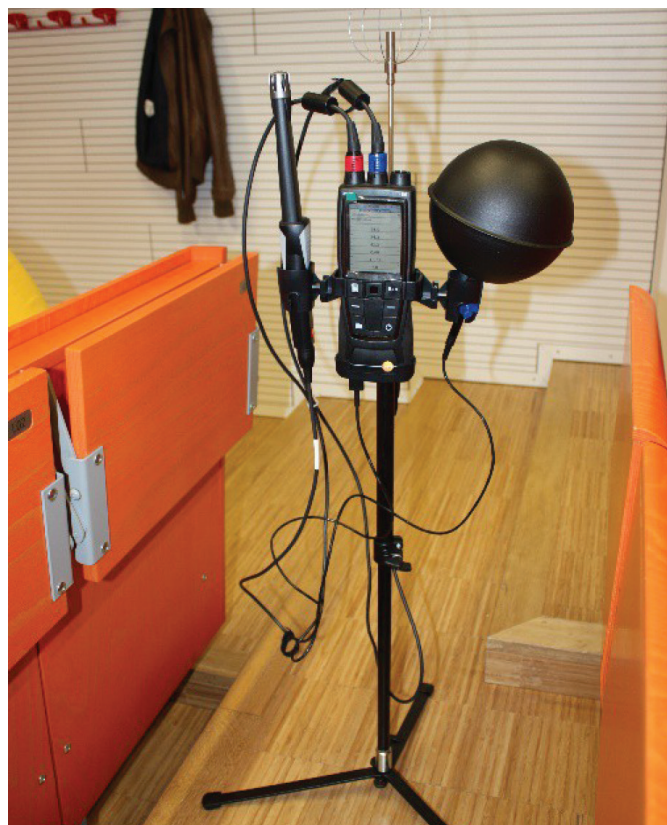


Fig. 3 Measurement of CO_2 concentration parameters and PMV index

3.2 Results and Analysis of the Indoor Climate Research

Fig. 4 shows the CO_2 concentration values at the four standpoints, i.e., A, B, C, D, at a height of 1.1 m above the floor level. At all the standpoints, the CO_2 concentration values were of acceptable sizes. The CO_2 concentration values were slightly higher in standpoints B and C compared to standpoints A and D, which were caused by the higher concentration of the sitting students. The acceptable values of the CO_2 concentration demonstrate the need for a forced ventilation system in small auditoriums. Then the acceptable values of the CO_2 concentration proved the suitability of the air distribution system in small auditoriums. The best location for air supply vents is in the sloped floor.

Fig. 5 shows the values of the PMV index at a height of 1.1 m above the floor level at standpoints A, B, C and D. The PMV index value was optimal and balanced at all the standpoints. The PMV index values at standpoints C and D reached optimal values, although they were slightly lower. The maximum value of the PMV index 0.3 was measured, which we can assess as a reasonable result. The results indicated the appropriateness of the use of the forced ventilation system and the correct location of the vents for the air supply in the sloped floor.

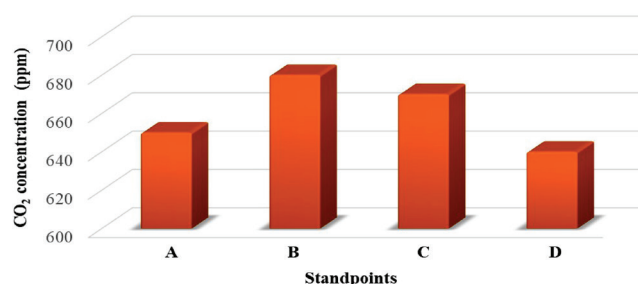


Fig. 4 Values of CO_2 concentration at the A, B, C and D standpoints

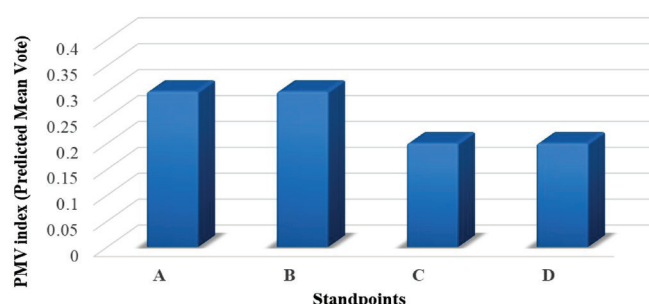


Fig. 5 Values of PMV index at A, B, C and D standpoints

4 CONCLUSIONS

The first experimental measurements of the CO_2 concentration were carried out in a small auditorium in an old building. They proved that natural ventilation for a small auditorium is very unsuitable. Extremely high unacceptable CO_2 concentration values were measured. From the point of view of the attention and concentration of students, the natural ventilation for a small auditorium was inadequate, significantly disrupted

lecture, and shortened its duration due to an unplanned break after forty minutes.

The second experimental measurements of the concentration CO_2 were made in a small auditorium in a newly-built building. They clearly showed the need for a forced ventilation system. The significant difference in the CO_2 concentration values between the two measurements shows the need to apply a high-performance, forced ventilation system and the need to renovate old small auditoriums. The acceptable values of the CO_2 concentration proved the suitability of an air distribution system from below upwards in the small auditorium. The acceptable values of the CO_2 concentration proved that it is advisable to place the vents for the air supply in a sloped floor. The values measured of PMV index proved the same.

The experimental measurements in the small newly-built auditorium showed that the air distribution system is very important, it is appropriate to the air distribution from below upwards. It is characteristic of this air distribution system that the air velocity in the vents for the air supply can be very low and that a sitting student does not feel a draught. With this air distribution system, it is most suitable to apply a sloped floor. This means that the architectural solution must be adapted to the optimal air distribution system.

The experimental measurements in the small, newly-built auditorium also demonstrated the great importance of an intel-

ligent control system. The air exchange can be set according to the CO_2 concentration. In this way, it is possible to ensure that the CO_2 concentration never falls below the required value. Before and after a lecture, it is necessary to ventilate the small auditorium intensively. This can be set by the control system.

Today, it is not possible to rely on natural ventilation in an auditorium. Students have to move to other auditoriums; a lecturer has other duties to perform; and it is not possible to have extra personnel who will take care of the ventilation in the interior of small auditoriums. High-quality ventilation is most important in the winter, when a greater number of different germs can enter the air in the small auditorium. The experimental measurements have shown that a forced ventilation system, a proper air distribution system, and a high-quality intelligent control system are essential for high-quality ventilation in a small auditorium.

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THE ANGLE OF THE SHEAR RESISTANCE OF DANUBE GRAVEL DERIVED FROM THE DYNAMIC PENETRATION TEST

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Abstract

The dynamic penetration test was used to investigate the angle of shear strength of Danube gravel. The correlations for its determination are available in the standards and literature. From the data set of the results, the characteristic value of the angles of shear strength was derived according to the recommended statistical procedure from the second generation of Eurocode 7. The characteristic values of the angle of shear strength were evaluated for well-graded gravels, poor-graded gravels, and gravels with fine-grained particles of the Danube. The correlations, which were based on field tests and considered the genesis of the soil, were compared with laboratory test results and a table of the values from the Slovak standard, which are used for purposes of comparison.

In our research we compared the characteristic field values of the angle of shear strength obtained in the laboratory to observe the effect of their genesis on the appropriateness of the correlations used. We conclude with recommendations for the regional correlations and suggest values of the angle of shear strength for the classes of gravels tested.

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Key words

- Characteristic value,
- Angle of shear strength,
- Danube gravel,
- Dynamic penetration test.

1 INTRODUCTION

Dynamic penetration (DP) belongs among widely used in situ soil tests in geotechnical engineering. Together with the standard penetration test (SPT), it belongs to the category of penetration tests that are driven into the soil by dynamic force. The two tests, along with the cone penetration test (CPT), measure the resistance of soil against the penetration of a rod. In Slovak geotechnical practice, heavy dynamic penetration is widely used in engineering-geological surveys of coarse-grained soils.

It is possible to derive specific geotechnical parameters of coarse-grained soils using the appropriate correlations for test results. These correlations have international and local (Slovak)

significance. Correlations applicable internationally are published in various standards (STN EN 1997-2:2008, DIN 4094-3:2002, EN ISO 22476-2, PN-B-04552:2002, STN 72 1032: 1997), which mainly focus on correlations for relative density (I_D). These correlations are direct, which means that they express a direct relation between the results of DP and the relative density of coarse-grained soil. The applicability of these correlations was studied by Vrettos and Papamichael (2018), who concluded that correlations are most reliable when evaluating the relative density of medium-dense soils.

The standards also mention an indirect correlation for the angle of shear strength, which is specifically based on the relative density of the soil, but not directly based on the DP results. Vrettos and Papamichael (2018) have stated that the use of this

correlation brings about uncertainties and instead recommend the use of direct correlations.

The correlations between the relative density and angle of shear strength of coarse-grained soils have also been described by other authors (Schnaid, 2009; Carter, Bentley, 2016; Sanglerat, 1972, Bolton, 1979; Robertson, and Hughes 1986). These correlations were derived from the number of N_{SPT} blows of the Standard Penetration Test. Lobo et al. (2009) expressed the relationship between SPT and ϕ_{ef} as dependent on the soil's stiffness, which is thus applicable in a constant N profile and has yet to be verified for a general procedure. Hatanaka and Uchida (1997) investigated the ϕ_{ef} of sands using laboratory tests, which were compared with the results of SPT tests. They derived a correlation from the comparison between N_{SPT} and ϕ_{ef} of sands with a validity range of 3.5–30 N_{SPT} .

Other direct correlations between the results of the DP test and selected soil parameters are used in Slovakia (Matys et al., 1990). These correlations are derived from the results of dynamic penetration resistance – q_{dyn} [MPa], which considers the effect of the groundwater level, friction on the rod, and the increasing weight of the probe with increasing depth. Most of the published correlations state a limitation of applicability based on the soil class. However, a few correlations also specify the genesis of the soil.

From the results of the geotechnical parameters, it is necessary to derive the representative value that is essential for the design of geotechnical structures. According to the second generation of Eurocode 7 (EN 1997-1:2024), the representative value can be represented by the nominal or characteristic value. The procedure for the determination of the characteristic value was mentioned in the previous generation of EC 7 (EN 1997-1:2005), which has been discussed by various authors. Orr (2019) stated that in addition to a statistical evaluation, previous comparable experiences should also be considered in an evaluation of the characteristic value. Prästings et al. (2019) said that the classical statistical procedure of confidence intervals does not consider the uncertainties of geotechnical properties. Schneider (2010) and Frankovská (2016) investigated the applicability of the determination procedure of the characteristic value published in Eurocode 7 (EN 1997-1:2005). The

authors pointed out that rock and soil properties are quantitatively described by one parameter, i.e., the characteristic value, the definition of which was not clear in EC 7. They stated that it is not possible to follow the same statistical rules as for man-made materials, and it is necessary to know the basic statistical characteristics of the data set.

The new generation of EC 7 (EN 1997-1:2024) describes the procedure for an evaluation of the characteristic value considering the basic statistical characteristics such as the mean, standard deviation, or coefficient of variation. In this article, the method from the second generation of EC 7 has been applied to derive the characteristic value of the angle of shear strength for Danube gravel, which was derived from the results of dynamic penetration tests.

To study the appropriateness of the correlations used with respect to genesis, the results of the angle of shear strength were compared with the laboratory results, other correlations, and the table (nominal) values from canceled Slovak standard (STN 73 1001:1987), which are still used as a comparable experience.

2 MATERIALS AND METHODOLOGY

The Dynamic Penetration Test was used for the examination of Danube gravels as a part of engineering-geological investigations conducted in various parts of Bratislava. The locality belongs to a geotectonic unit that is mostly formed by Neogene and Quaternary soils. Quaternary soils are present in the form of the sand-gravel sediments of the Danube River.

In this region, the river has predominantly accumulative and sedimentary activity, which has affected the shape and properties of river gravels and sands. Due to the river's activity, the gravels are characterized by their smooth and fine particles (Fig. 1), which are well-rounded and 1 – 5 centimeters in size.

In our research we focused on analyzing the angle of shear strength of the Danube gravels G1/GW, G2/GP, and G3/GW, which are the most representative gravel classes in the area. The well-graded G1/GW gravels contain 63 – 76 % of gravel particles and grading of the particles at d_{50} varies from 5 to 10 millimeters (Fig. 2).



Fig. 1 Samples of Danube gravels

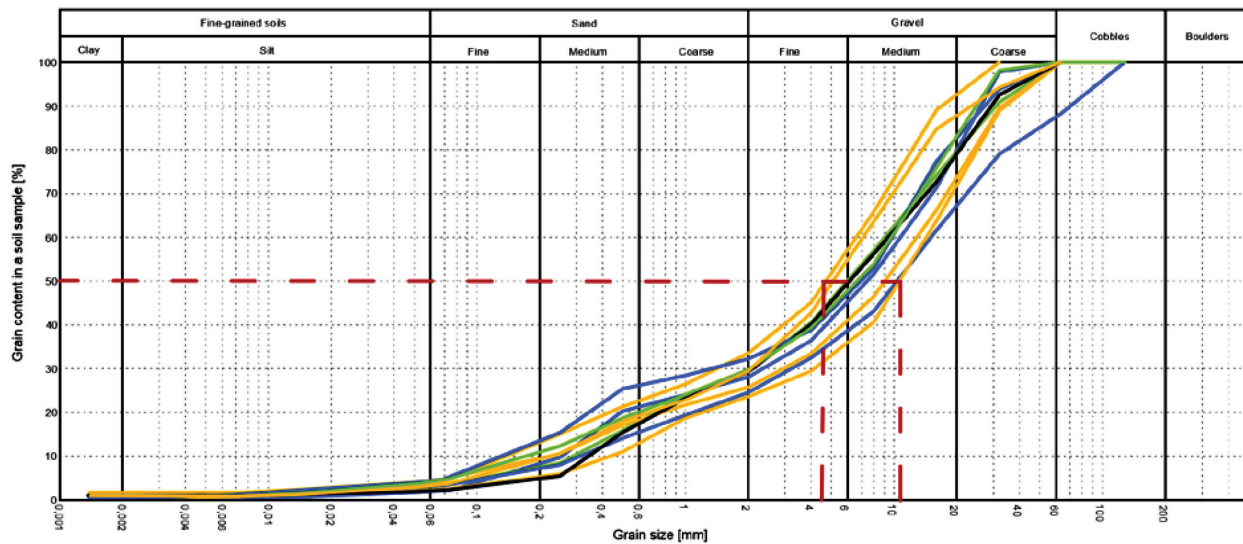


Fig. 2 Grain size distribution of well-graded gravels (G1/GW)

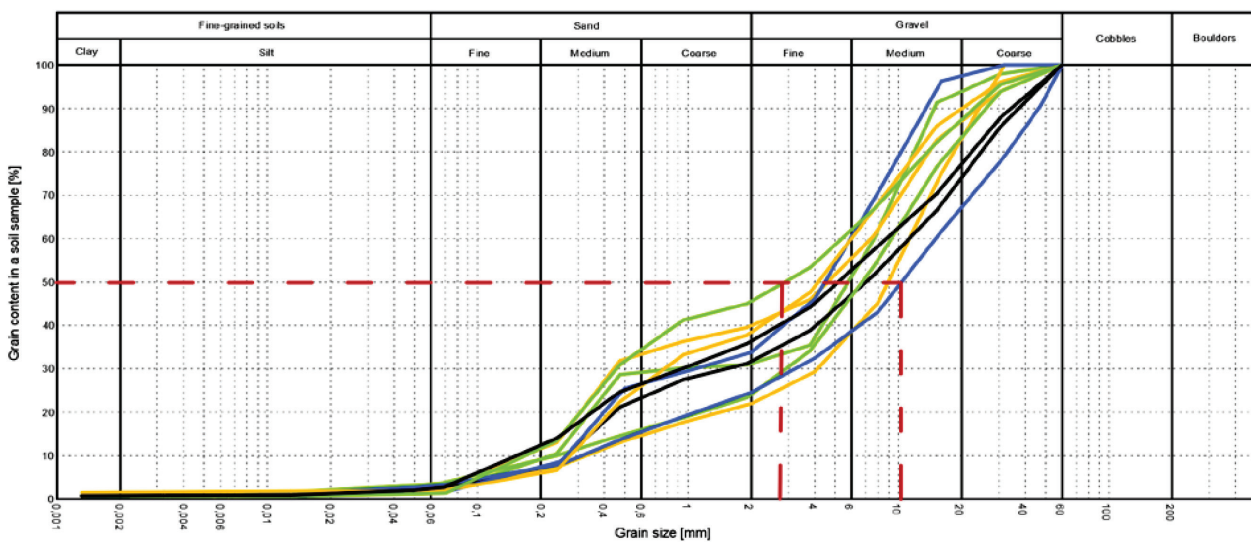


Fig. 3 Grain size distribution of poorly-graded gravels (G2/GP)

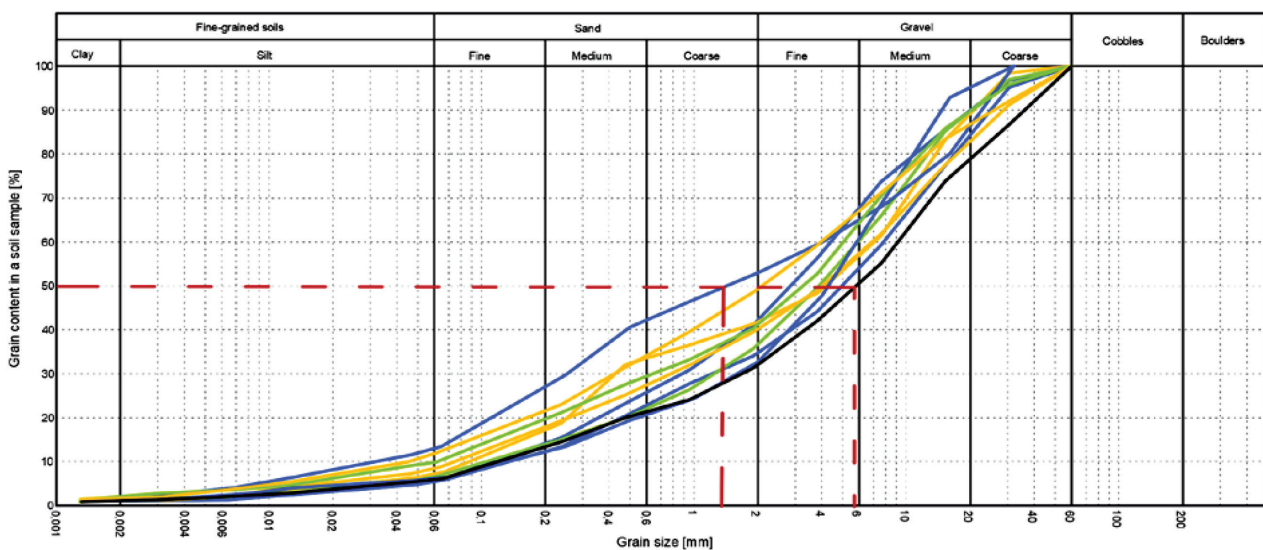


Fig. 4 Grain size distribution of gravels with fine-grained soil (G3/G-F)

Poorly-graded gravels contain about 54 – 77% of gravel fractions, and the grading of the particles at d_{50} is from 3 to 10 millimeters (Fig. 3). The content of the gravel fraction in gravels with fine-grained particles (G3/G-F) varies from 45 – 70%. The grading of the particles at d_{50} is 1.5 – 6 millimeters (Fig. 4).

2.1 Determination of the angle of shear strength from field tests

The angle of the shear strength of the Danube gravels was derived using various correlations (Tab. 1, eq. 1). According to Švasta and Obert (Matys et al., 1990), the correlations express a direct relation between the soil property and the results of the dynamic penetration test (N , q_{dyn}). Other correlations published in standards (EN 1997-2:2008) and non-Slovak literature (Bolton, 1979; Robertson and Hughes, 1986) estimate the angle of shear strength based on the relative density of the coarse-grained soils. Eurocode 7 (EN 1997-2:2008) presents correlations that are directly applicable to the dynamic penetration test results. Bolton (1979) and Robertson and Hughes (1986) describe the correlation valid for the Standard Penetration Test (SPT). Using the relation between SPT and DPT published by Carter and Bentley (2016), the correlation for SPT can be applied to the DPT as well. Carter and Bentley determined that N_{SPT} (blows/ 300 mm) is approximately equal to $2 \times N_{DPH}$ (blows/ 100 mm). However, the correlation can be used for all the variations of the DPT test as the authors expressed the general formula for recalculating N_{SPT} to N_{DPT} , considering the parameters and depth of penetration of the probe.

The British standard (BS 8002:1994) describes the correlation that expresses the relation between the angle of shear

strength and the results of blows – N , taking into consideration the shape of the particles and grading of the coarse-grained soils. In general, the correlation is applicable to all classes of gravels or sands and was established based on a comparison.

$$\varphi = 30^\circ + \left\{ \begin{array}{l} 0 - R \\ 2 - SA \\ 4 - A \end{array} \right\} + \left\{ \begin{array}{l} 0 - U \\ 2 - MG \\ 4 - WG \end{array} \right\} + \left\{ \begin{array}{l} 0 - N < 10 \\ 2 - N = 20 \\ 6 - N = 40 \\ 9 - N = 60 \end{array} \right\} \quad (1)$$

Key:

R – round
SA – sub-angular
A – angular

U – uniformly graded.
MG – moderately graded.
WG – well graded.

2.2 Laboratory determination of the angle of shear strength

The large-scale shear box test was used to verify the shear parameters (φ_{ef}) of the gravel samples, i.e., dense gravels of class G1, G2, and G3. A series of tests on a large-scale Shear-matic 300 shear box test was carried out in the laboratory of the Department of Geotechnics. Due to the dimensions of the shear plane, i.e., 300x300 mm, it is suitable for testing coarse-grained soils with grains up to 15 mm in size. The tests were performed in accordance with STN EN ISO 17892-10:2018.

Each test consisted of a series of 3 to 4 shearings, each performed with a different effective normal stress σ_{eff} . The velocity of the shearing was set up at 1 mm/min.

Tab. 1 Correlations between the DPT and angle of shear strength of coarse-grained soils

Author	Soil	φ_{ef} derived from N_{10}		
		N_{10}	φ_{ef} (°)	
STN 72 1032	Gravels	< 3	< 30	
		3 – 6	30 – 35	
		6 – 17	35 – 40	
		17 – 30	40 – 45	
		> 30	> 45	
Švasta	Correlation	Coefficients		
	[°]	p	24	
		r	0,16	
EN 1997-2:2008	Soil	φ_{ef} (°) derived from I_{D}		
		<i>Loose</i>	<i>Medium dense</i>	<i>Dense</i>
	Fine-grained sand/ sand, and gravel ($C_{\text{u}} < 6$)	30	32.5	35
	Sand and gravel ($6 < CU < 15$)	30	34	38
Bolton (1979)	Well-graded sand or gravel	-	-	55 ¹ /35 ²
Robertson and Hughes (1986)		-	-	40
Note: ¹ Peak angle of shear strength ² Angle of shear strength at critical state				

Note: ¹ Peak angle of shear strength ² Angle of shear strength at critical state

2.3 Representative value of the geotechnical parameters

The determination of the representative value of geotechnical parameters is an important step in the design of geotechnical parameters. According to the new generation of Eurocode 7 (EN 1997-1:2024), the representative value can be expressed as a nominal, conservative estimate of the value affecting the limit state or characteristic value. In our research, we used the characteristic value of the angle of shear strength of the Danube gravels as a representative value.

The characteristic value (X_k) is defined as a value affecting the limit states that corresponds to a specific quartile of the assumed statistical distribution of a specific soil property. In the second generation of Eurocode 7, there is a given formula (eq. 2) for the calculation of the characteristic value:

$$X_k = X_{mean} [1 \mp k_n V_x] = X_{mean} \left[\frac{1 \mp k_n \sigma_x}{X_{mean}} \right] \quad (2)$$

where

- X_{mean} average of the measured values of the soil property.
- V_x coefficient of the variation of the measured values of the soil property.
- k_n coefficient depending on the frequency of the measured values (n) of the soil property.
- σ_x standard deviation of the measured values of the soil property.

The procedure for the evaluation of the characteristic value is suitable for known/ assumed and unknown coefficients of the variation. According to the case of V_x , there are formulas given for the evaluation of coefficient k_n (Tab. 2), which depend on two distribution functions – normal and Student's t distribution function.

Coefficient N_{95} (in Tab. 2) stands for a 95% confidence level and infinite degrees of freedom regarding the normal distribution. Coefficient $t_{95,n-1}$ is evaluated for a 95% confidence level and $n-1$ degrees of freedom regarding the Student's t distribution, n being the number of samples used for an evaluation of X_{mean} .

Tab. 2 Determination of coefficient k_n with respect to the type of the coefficient of the variation

Cases	Known & assumed V_x	Unknown V_x
Estimate of the mean value	$k_n = N_{95} \sqrt{\frac{1}{n}}$	$k_n = t_{95,n-1} \sqrt{\frac{1}{n}}$
Estimate of the inferior or superior value (5 or 95% quartile)	$k_n = N_{95} \sqrt{1 + \frac{1}{n}}$	$k_n = t_{95,n-1} \sqrt{1 + \frac{1}{n}}$

The aim of our research was to determine the critical value of the selected soil properties. Therefore, we evaluated the characteristic value as the lower quartile of the corresponding statistical distribution. The coefficient of variation was not previously specified. It was evaluated during the statistical analysis, and the equation for the evaluation of coefficient k_n was chosen accordingly, based on the Student's t distributional function.

3 RESULTS

An analytical determination of the angle of shear strength for the Danube gravels G1, G2, and G3 was done using correlations according to Švasta and STN 72 1032 (Tab. 1), which are widely used in Slovakia. The angle of shear strength was determined from the results of N_{10} and q_{dyn} of the DPH test. The data set was classified respectively according to the relative density of the gravels.

3.1 Well-graded gravel (G1/GW)

For well-graded gravels, the data set analyzed consisted of 16 results (N_{10} , q_{dyn} , φ_{ef}) which were categorized into two classes of relative density, i.e., medium dense and dense (Tab. 3). According to the correlation in STN 72 1032, the angle of shear strength of G1 is in a range of $25^\circ - 40.8^\circ$ (medium dense to

Tab. 3 Statistics of data for G1/GW

Variable	Relative Density	N	Mean	Standard Deviation	Min.	Median	Max.
N_{10} [-]	Medium Dense	10	8.72	4.78	2.00	9.86	15.24
	Dense	6	20.81	4.40	16.50	19.22	28.17
q_{dyn} [MPa]	Medium Dense	10	9.46	3.58	3.05	9.27	14.11
	Dense	6	22.25	5.84	15.61	21.75	29.37
φ_{ef} [°] (STN 72 1032)	Medium Dense	10	31.15	3.47	25.00	32.43	35.12
	Dense	6	37.77	1.95	35.75	37.11	40.79
φ_{ef} [°] (Švasta)	Medium Dense	10	34.01	2.45	28.69	34.22	36.66
	Dense	6	39.27	1.68	37.25	39.27	41.22

dense). Using Švasta's correlations, the φ_{ef} of well-graded gravels has higher values, i.e., 28.7° - 41.2° (medium dense to dense).

The distribution of the results of the angle of shear strength for well-graded gravel derived by both correlations (Švasta, STN 72 1032) is graphically shown in Fig. 5.

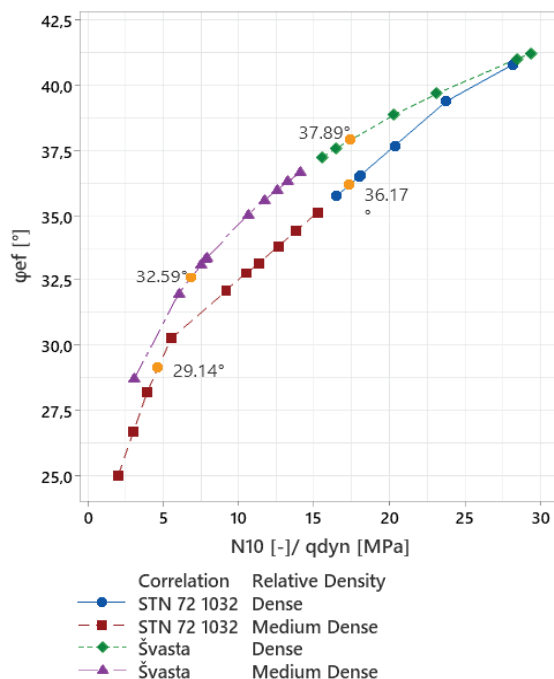


Fig. 5 Angle of shear strength of G1/GW derived by Švasta and STN 72 1032

In the same figure the representative value of φ_{ef} , which was evaluated separately according to each correlation and relative density respectively (marked with orange dot and number), is also denoted. From a comparison of the respective values (Tab. 3 and Fig. 5), the difference between the mean and characteristic values of φ_{ef} varies from 1.4° to 2° .

Tab. 4 Statistics of data for G2/GP

Variable	Relative Density	N	Mean	Standard Deviation	Min.	Median	Max.
N_{10} [-]	Loose	9	2.60	0.72	2.00	2.29	4.27
	Medium Dense	15	10.81	3.16	4.82	10.92	15.09
	Dense	15	20.42	3.82	15.06	19.79	27.08
q_{dyn} [MPa]	Loose	9	4.74	2.38	2.12	4.78	7.69
	Medium Dense	15	12.09	2.17	7.29	12.01	14.32
	Dense	15	21.59	4.93	15.22	20.87	28.30
φ_{ef} [°] (STN 72 1032)	Loose	9	24.83	4.56	20.00	22.86	32.11
	Medium Dense	15	37.02	1.78	33.03	37.24	39.13
	Dense	15	41.29	1.49	39.12	41.08	43.88
φ_{ef} [°] (Švasta)	Loose	9	30.27	2.66	27.07	30.83	33.27
	Medium Dense	15	35.68	1.12	32.98	35.72	36.74
	Dense	15	39.11	1.45	37.10	39.02	40.97

3.2 Poor-graded gravel (G2/GP)

The data set for poor-graded gravel contain 39 results of N_{10} and q_{dyn} , from which the φ_{ef} was derived. The results were categorized according to relative density to N_{10} , q_{dyn} , and the φ_{ef} of loose, medium dense, and dense gravels G2 (Tab. 4).

Using the correlation in STN 72 1032, the angle of shear strength of the loose G2 is determined to be in the range of 20° – 32.1° .

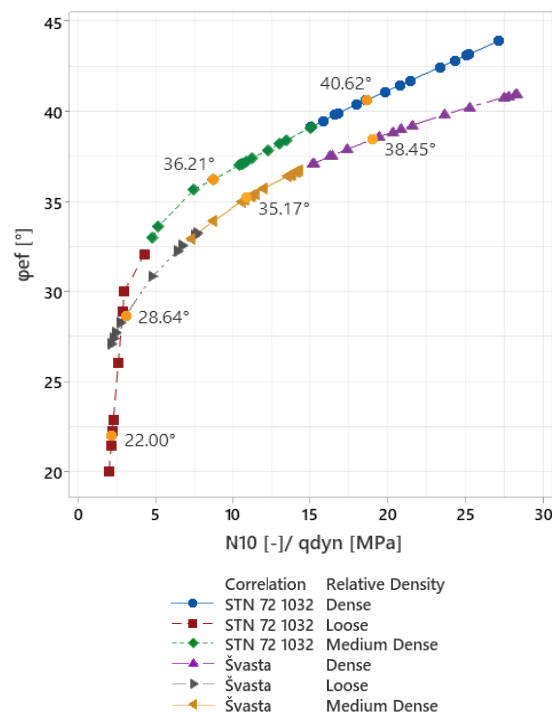


Fig. 6 Angle of shear strength of G2/GP derived by Švasta and STN 72 1032

For medium to dense gravels, the φ_{ef} is in a range of 33° – 43.9° . Using Švasta's correlations, the φ_{ef} of loose, poor-graded gravel is higher, i.e., from 27.1° to 33.3° . The values of φ_{ef} for a medium dense to dense state are 32.9° – 40.9° (Tab. 4).

A graphic representation of the φ_{ef} derived from both correlations (Švasta, STN 72 1032) is shown in Fig. 6. The corresponding representative value of the angle of shear strength is marked with an orange dot and number. The comparison of the results (Tab. 4 and Fig. 6) shows that the difference between the mean and the characteristic value of φ_{ef} for loose gravels G2 is 2.8° . For medium and dense gravels, the difference between the values varies from 0.6° to 0.8° .

3.3 Gravel with fine grained soil (G3/G-F)

The data set analyzed for gravels with fine-grained particles contains 51 results of N_{10} , q_{dyn} , and φ_{ef} , which were organized according to their relative density into the results of the medium dense and dense gravels G3 (Tab. 5). According to the correlation in STN 72 1032, the angle of shear strength for G3 is evaluated to be from 30.6° – 44.8° (medium dense to dense).

According to Švasta's correlation, the φ_{ef} of G3 gravels is from 32.6° to 42.4° (medium dense to dense) and indicates lower values of φ_{ef} than the correlation in STN 72 1032 (Tab. 5, Fig. 7).

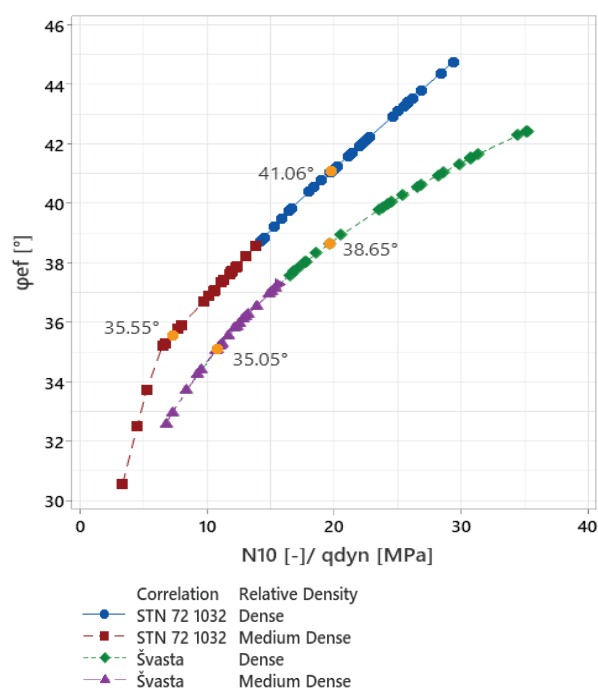


Fig. 7 Angle of shear strength of G3/G-F derived by Švasta and STN 72 1032

Tab. 5 Statistics of data for G3/G-F

Variable	Relative Density	N	Mean	Standard deviation	Min.	Median	Max.
N_{10} [-]	Medium Dense	20	9.62	3.04	3.33	10.53	13.83
	Dense	31	21.18	4.31	13.84	21.42	29.35
q_{dyn} [MPa]	Medium Dense	20	11.91	2.61	6.75	12.24	16.71
	Dense	31	22.86	7.26	8.35	23.85	35.27
φ_{ef} [°] (STN 72 1032)	Medium Dense	20	36.35	2.05	30.56	37.06	38.56
	Dense	31	41.56	1.70	38.56	41.70	44.75
φ_{ef} [°] (Švasta)	Medium Dense	20	35.56	1.31	32.57	35.83	37.66
	Dense	31	39.31	2.17	33.71	39.87	42.44

Tab. 6 Comparison of φ_{ef} values determined by the laboratory, field tests, and table values

Soil	Relative density	φ_{ef} [°]						
		Lab. Tests	Field tests					Table values (STN 73 1001)
			Švasta	STN 72 1032	BS 8002	EC7/ DIN	Bolton/ Robertson	
G1/GW	Medium dense	-	32.59	29.14	34	36	-	38.5
	Dense	40.9/42.8	37.89	36.17	38	40	55/35	41.5
G2/GP	Loose	-	28.64	22.00	30	30	-	-
	Medium dense	-	35.17	36.21	32.5	34	-	35.5
	Dense	41 – 48.5	38.45	40.62	35	38	-	38.5
G3/G-F	Medium dense	-	35.05	35.55	34	36	-	32.5
	Dense	44 – 51.9	38.65	41.06	55/35	38	40	35.5

A graphic representation of the φ_{ef} results derived by both correlations (Švasta, STN 72 1032) and their representative values are shown in Fig. 7. From a comparison of the results (Tab. 5 and Fig. 7), the difference between the mean and characteristic values of φ_{ef} varies from 0.5° to 0.8° , which is not significant.

Tab. 6 summarizes the representative (characteristic) values of the analytical determination of φ_{ef} from the results of the N_{10} and the q_{dyn} of the heavy dynamic penetration test. These results are compared with the laboratory results, other correlations, and the table (nominal) values, which were published in the canceled Slovak standard STN 73 1001. Despite the fact that the Slovak standard has been canceled, these table (nominal) values are still used for purposes of comparison.

It can be seen from the comparison in Tab. 6 that for the *loose gravels*, there is no recommendation of φ_{ef} in STN 73 1001:1987. However, according to the analytical evaluation using correlations by Švasta, STN 72 1032, and BS 8002:1994, the angle of shear strength of G2 gravels reaches a minimal value of 25° to 30° . The correlation in Eurocode 7 and the German standard (EN 1997-2:2008, DIN 4094-3:2002) also specifies 30° for the angle of shear strength of loose gravel.

For *medium dense and dense gravels* of the class G1, the angle of the shear strength derived by Švasta and STN 72 1032 is lower than the φ_{ef} determined by other correlations and the table (nominal) values. The laboratory test indicates higher values as well. The angle of shear strength of medium dense and dense gravels G3 is greater than the φ_{ef} of G1 gravels.

Based on the analysis and a comparison of the results with the non-Slovak correlations, we recommend the correlation according to Švasta for the evaluation of the angle of the shear strength of the Danube gravels. It is also appropriate for the medium dense and dense Danube G1, G2, and G3 gravels when reevaluating the table (nominal) values of φ_{ef} . For loose gravels, we recommend using 30° for the angle of shear strength; it represents a conservative estimation.

4 CONCLUSION

In the article, we analyzed the angle of the shear strength of Danube gravels derived from the results measured of a dynamic penetration test. The correlations that were used for the analytical derivation of the angle of shear strength have regional (Slovak) applicability (Matys et al, 1990). These correlations, unlike the correlations in standards (STN EN 1997-2:2008, DIN 4094-3:2002, EN ISO 22476-2, PN-B-04552:2002, STN 72 1032: 1997; BC 8002:1994), express a direct relation between the results of the dynamic penetration and the required soil property. Furthermore, the results of the dynamic penetration resistance consider physical aspects, such as the influence of the groundwater level, friction on the rod, and the increasing weight of the probe with depth, and are therefore more accurate (Matys et al, 1990).

For the design of geotechnical structures, it is necessary to know the representative value of the geotechnical parameters. In the second generation of Eurocode 7 (EN 1997-1:2024), the representative value could be either a nominal or characteristic value. For the determination of the characteristic value, the

new Eurocode 7 gives a recommended procedure that works with the basic statistical characteristics. This method is clearly described and easy to follow for the average user.

In following the above, we determined the characteristic value of the Danube gravels from the data set of the derived angles of shear strength. The results and characteristic value of the angle of shear strength were sorted according to the relative density of the gravel. Total of three classes of gravels (G1/GW, G2/GP, and G3/G-F) were analyzed. The characteristic value was determined for each class and stage of the relative density of the soil. The characteristic values of the angle of shear strength were compared with the table (nominal) values from the canceled Slovak standard STN 73 1001:1987, which are still used for purposes of comparison. It must be said that the table values and most of the available correlations between the DP results and gravel properties are generalized and do not consider the genesis of the soil (roundness, shape, size, roughness of the grain surface).

Therefore, in order to observe the effect of the genesis on the appropriateness of the correlations used, the table values, and the parameters evaluated, we compared the results of the characteristic values of the angle of shear strength obtained with the laboratory results and other correlations. Based on the analysis of the loose gravels, we recommend values that are conservatively selected, as STN 73 1001:1987 does not list any values of the angle of the shear strength for loose coarse-grained soil.

For the medium dense and dense gravels, the most significant difference between the characteristic values of the angle of shear strength and the recommended values in STN 73 1001:1987 is shown for the G1 and G3 gravels. The values of the angle of shear strength of G1 gravels derived from DP using the Slovak correlations are lower than the values determined by other correlations (BS 8002:1994; EN 1997-2:2008; DIN 4094-3:2002; Bolton, 1979, Robertson and Hughes; 1986) and the table values. The angle of the shear strength of the G3 gravels derived from DP using Slovak and foreign correlations is greater than the angle of shear strength of the G1 gravels. This is also confirmed by the results of the laboratory tests. Therefore, it would be appropriate to reevaluate the table values for all the gravel classes analyzed.

Acknowledgments

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THE USE OF VERTICAL GARDENS AS A NETWORK OF URBAN NAVIGATION ELEMENTS WITH A POSITIVE IMPACT ON BIODIVERSITY AND MICROCLIMATE IN A DENSE URBAN ENVIRONMENT

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Abstract

Horizontal green areas are becoming valuable, yet extruded spots, and their constant elimination at the expense of commercial and housing developments is disrupting green infrastructure. Interlocked existing urban structures seldom allow for the creation of parks and larger green open spaces, which could provide a habitat for a wide range of insects, birds, and other small animals. The lack of green spaces causes urban heat islands (UHI) over the cities. Vertical gardens are alternative solutions for urban areas, where it is not possible to increase biodiversity by creating new horizontal green areas. They effectively utilise surfaces that would otherwise be unused and transform them into effective green infrastructure elements.

Using the example of the historical city district of Bratislava, i.e., Old Town (Staré Mesto) in Slovakia, we present an alternative approach for resolving the lack of green areas. A case study was conducted to create a map of all the potentially suitable areas for the construction of vertical gardens. We divided these spots into three groups, i.e., walls, columns, and free-standing structures and proposed construction methods for each type. Regarding the placement of potentially suitable areas along the roads connecting important transport hubs with the city centre, we created an original proposal for a district-wide network of vertical gardens as orientation elements for tourists and residents with significant aesthetic merit as well as ecological and hygienic values.

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Key words

- Vertical gardens,
- Green walls,
- Densely urbanised cities,
- Urban heat island mitigation,
- Urban design,
- Living walls.

1 INTRODUCTION

The constant expansion of densely built-up areas caused by intensive urbanisation eliminates green areas and contributes to a loss of biodiversity and the overheating of cities compared to surrounding rural areas. Developing various construction and innovative methods could mitigate some of these issues caused by urbanisation. The absence of conventional green spaces or locations for creating new ones is leading us to find

alternative solutions to urban greening (Susca *et al.*, 2022). One of those measures involves vertical gardens or living wall constructions. This approach is still relatively new, with only limited research regarding its benefits to the urban environment and its application in local and national policies (Liberalesso *et al.*, 2020). A review study by Balany *et al.* (2020) shows that only 4.3% of reviewed papers examined living walls as a green infrastructure strategy. Moreover, another analysis of green infrastructure incentives in 113 international cities shows no par-

ticular living wall promotion policy (Liberalesso *et al.*, 2020). Considering the relatively significant number of building surfaces suitable for creating living walls and their presence on the street level, such a measure shows great potential in mitigating heat stress, increasing biodiversity, and creating a better urban environment (Emilsson *et al.*, 2007).

The aim of this paper is to offer a brief description of current findings regarding the positive effects of vertical gardens to mitigate urban heat islands (UHI), and improve energy efficiency, microclimates, and greywater treatment (Ruiz-Valero *et al.*, 2023). Moreover, we propose a strategy that uses bare facades, columns, walls, and other existing urban structures and transforms them into effective green infrastructure elements. An added value of the research is our original proposal to use vertical gardens as a network of orientation points from the main traffic hubs, i.e., railway and bus stations, public transport interchanges, and passenger port, to the historical centre of Bratislava, thus creating an urban navigation system.

1.1 Positive impact of vertical gardens

Densely urbanised areas lack biodiversity; hence, vertical gardens provide opportunities to create habitats for birds, butterflies, bees, and other small insects (Elgizawy, 2016). When designed correctly, vertical gardens could have a significant impact on improving biodiversity in urban areas (Thorpert *et al.*, 2022). Using bare structures with no ecological value and changing them into green sites could help create a network of connecting stops between larger green areas. Studies indicate that when combined together, these small patches could have a major positive effect on increasing biodiversity (Vega and Küffer, 2021). The lack of green spaces allow hot air to accumulate above the city and it causes urban heat islands (UHI) over the cities. An urban heat island is caused by various factors, mainly heat absorption and its subsequent release by sealed surfaces such as roads, pavements and building facades (Taleghani, 2018). Plants can absorb solar radiation for their vital functions, which help the surrounding microclimate by evaporative cooling (Qin *et al.*, 2012; Majorošová and Mazancová, 2020) and an increase in albedo. Materials such as asphalt and concrete have an albedo of 0.1-0.4, respectively, while green walls have one of 0.7-0.85. With increasing albedo values, on a scale of 0-1, heat absorption, and subsequently emissions, decrease (Berardi *et al.*, 2014). Another secondary contributor to UHI is the use of air conditioning. The rise of urban temperatures can increase demands for cooling, thus contributing to greenhouse gas emissions. Furthermore, heat waste from air conditioning aggravates the UHI effect (Susca *et al.*, 2022) and increases the urban temperature by 0.5°C (de Munck *et al.*, 2013). Analyses show that installing green walls can reduce heating and cooling energy demands by 16.5% and 51%, respectively (Susca *et al.*, 2022). Moreover, research has shown that during the cooling season in a Mediterranean climate, green facades can have an energy efficiency of 34% (Pérez *et al.*, 2017) and living wall systems one of up to 59-66% (Manso *et al.*, 2021). Vegetation and substrate act as insulation, provide shading of the wall surface, and absorb solar radiation, thus alleviating overheating of building interiors and reducing cooling demands in the summer

(Shafiee *et al.*, 2020). On the other hand, winter field measurements have shown an increase of near-wall temperatures by 1.26°C (Daemei *et al.*, 2018) or even 2°C, compared to a bare control wall, thus proving efficiency as a passive energy-saving measure (Vox *et al.*, 2018).

Several studies have proved the positive impact of green walls on surface temperatures and surrounding microclimates, including a decrease in average urban temperatures by 1.37°C (Manso *et al.*, 2021). An experiment conducted in Catalonia, Spain, (Coma *et al.*, 2017) showed reductions of 16.5°C in the surface temperature on a south wall, 4.5°C on an east wall, and 6.5°C on a west wall. Hoelscher *et al.* (2016) reported decreases in exterior surface temperatures in Berlin by up to 15.5°C and by 1.7°C on interior. An experiment in Bari, Italy, shows a maximum surface temperature drop of up to 9°C on a south wall (Vox *et al.*, 2018), similar to an Illinois experiment by Susorova *et al.*, (2014). Other research from Thessaloniki, Greece, reports an average surface temperature reduction of 5.7°C, varying from 1.9°C to 8.3°C (Eumorfopoulou and Kontoleon, 2009). Perez *et al.* (2014) stated that the surface temperature decreased by 1.7°C to a maximum of 13°C.

According to Oquendo-Di Cosola *et al.* (2022), their results show different values for various climate types. Depending on the construction method, a wide range of values is presented even though measured under identical circumstances. In humid tropical climates, the reduction in the temperature varies between 1.1°C – 14°C for continuous green walls, when using materials such as felt, which supports roots and lets plants grow naturally, and from 6.1°C up to 20.8°C for modular green walls, which are formed by modular substrate-filled, pre-planted pot units. An even more significant decrease was observed in continental climates, where modular types showed a reduction between 8°C and 31°C, while continuous green walls mildly decreased the surface temperature by 0.8°C to 2.1°C. In a Mediterranean climate, both modular and continuous systems show reductions of up to 20°C, in an oceanic climate up to 15.5°C, and in semi-arid climates by 2.5°C (Oquendo-Di Cosola *et al.*, 2022).

Green walls also have potential in natural domestic greywater treatment. Roots and porous media could function as a bio-filter treating wastewater through oxidation, filtration, microbial assimilation, etc. (Manso *et al.*, 2021). Research shows an 80-90% removal of total suspended solids, a 30-50% removal of total nitrogen, a 15-30% removal of total phosphorus, a 20-80% removal of E-coli, a 30-70% removal of chemical oxygen demand, and more than a 90% removal of biological oxygen demands (Ghaididak and Yadav, 2013; Prodanovic *et al.*, 2017).

Other research shows that increasing humidity could lower temperatures in an urban environment. According to a study from Iran, relative humidity near a vegetated wall was up to 9.3% higher than the control bare wall and 7.3% higher at a 0.5m distance from the green wall (Shafiee *et al.*, 2020). Plants absorb carbon dioxide for photosynthesis, thereby producing oxygen and biomass, which also positively impacts a microclimate (Stemberg *et al.*, 2010). Foliage can retain dust particles that are subsequently washed down to drainage systems (Kuhn and Bass, 1999). Studies show that vegetated areas have only 10-15% dust particles compared to areas without any vegetation (North American Wetland Engineering, 1998).

Studies regarding the positive effect of green walls on the environment still need to be expanded, and they need to be the subject of further and more complex research. However, we can state that such a measure positively impacts the urban environment and its microclimate and increases the aesthetic and economic value of buildings (Fig. 5,6) (Hadba, 2017). They improve the overall appearance of cities and positively impact the human psyche (Sheweka and Magdy, 2011). One of the most valuable advantages of vertical gardens is their placement on vertical structures, which does not require taking up valuable land space. Instead, they give value to places that would otherwise be unused such as facades, walls, or columns. Vertical gardens also increase the economic value of buildings and neighbourhoods (Perini *et al.*, 2011). This technology is still a relatively rare and unusual sight to the public eye; hence, its limited use has been seen in an urban environment. Nonetheless, it could be perceived as a benefit since the unconventional appearance of such a design attracts tourists, making these buildings modern-day monuments and subsequently popularising green architecture.

2 METHODOLOGY

2.1 Selection of the research area

This paper is focused on the use of an alternative form of vegetation in a dense urban environment. It is a new way of green areas in the cities. Problems connected to dense urbanisation also occur in Slovakia.

We used the Old Town district of Bratislava, the capital of Slovakia, as a pilot case study for implementing green walls around the city centre. The methodology is based on our knowledge of the principles and values of vertical gardens. Densely locked-in historical urban structures in the Old Town do not allow for the creation of valuable new green spaces; on the other hand, these structures contribute the most to the UHI effect. Therefore, vertical gardens are ideal in this case, as they do not take up any space or land.

We defined area borders using the official cadastral boundaries of the Old Town district. Since it is a historical part of Slovakia's capital, the historical context of the area and its relations and values in the past were also analysed to create a design that does not interfere with its character. The use of green walls in the historical city centre and its surroundings could not only benefit biodiversity and the microclimate during hot summer days but also effectively increase the aesthetic value of abandoned or deteriorated buildings, which have a negative impact on the overall appearance of one of the most visited parts of the city and the whole country as well. We analysed the green environment of Old Town and focused on all the green spaces and elements present in this part of the city. Due to the lack of green areas, we chose the most densely urbanised area of the Old Town, i.e., the historical centre, as the main area of interest.

2.2 Data collection in Bratislava, Slovakia

Firstly, we got data from the Slovak Hydrometeorological Institute, which provided surface temperature analysis data collected in August 2018. The data were from the satellite

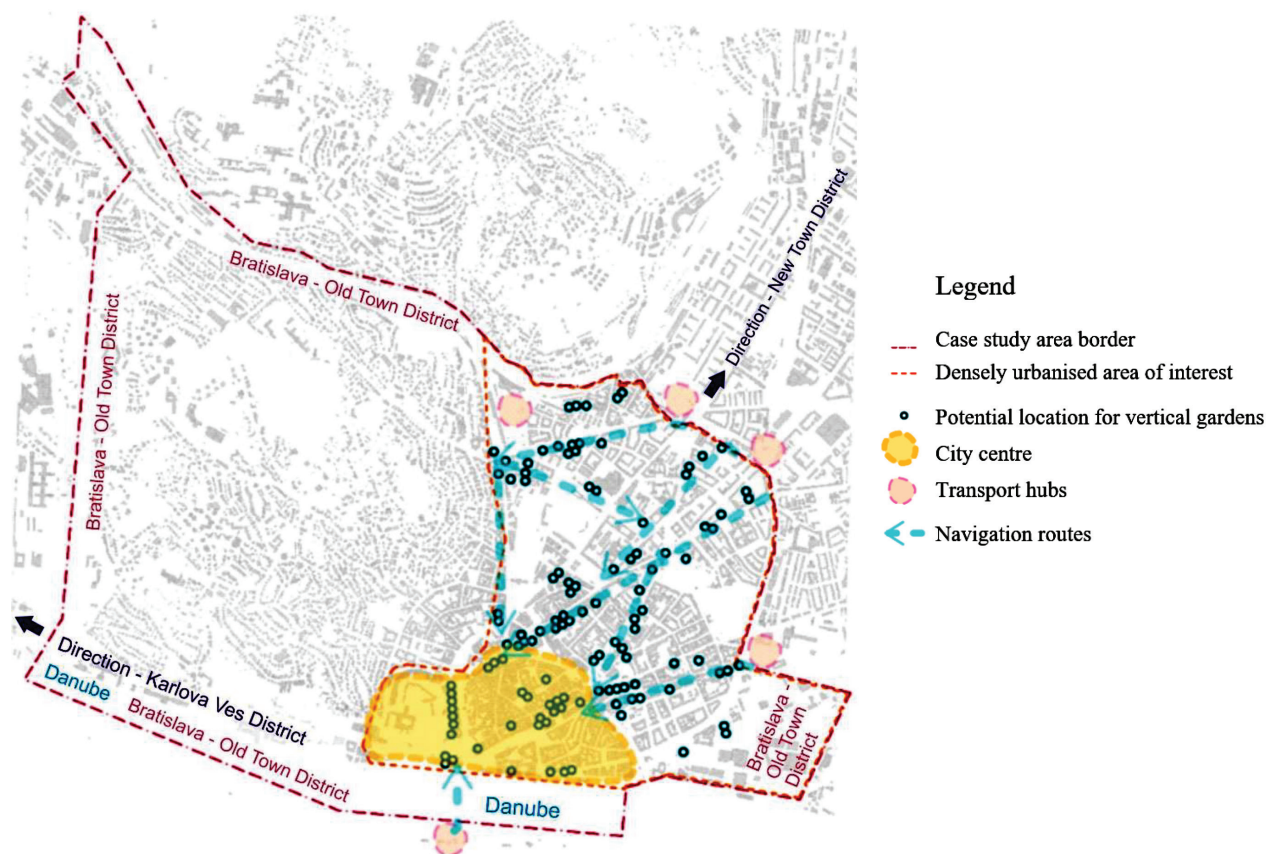


Fig. 1 Map of the Old Town Bratislava District, marking all the potential areas of interest.

Landsat 8, and the temperatures were measured at 0:00, 8:00, 12:00, 15:00 and 18:00. These measurements showed that the historical part of the Old Town is most prone to overheating, with up to a 10°C difference between the area of interest and rural areas (Holec, 2018).

As our next step, we collected data in situ and created a database and photo documentation of all the potential spaces suitable for vertical gardens. Those data collection field trips took place from September to November 2019. Then, we designed a map marking all the potential areas of interest (Fig. 1).

2.3 Choosing the type of construction

Lastly, we chose three different vertical structures that could be appropriate for the Bratislava environment and created three case studies that could be used as examples of design patterns for all the potential spaces featured on the areas of interest map. These three structures are: green walls, green columns, and free-standing green structures because there are many spots, where these tree types are applicable in Bratislava Old Town District.

Inspiration for the development of green walls comes from natural parasitic climbers and vines, especially in a tropical environment. These species are adapted to growth on rocks or other plants and receive nutrients from the atmosphere or rain. Vertical gardens can be divided into two primary groups, i.e., green facades and living walls (Brković Dodig *et al.*, 2019).

Green facades are the most affordable type of vertical gardens (Ottelé *et al.*, 2011). They are considered to be simple solutions created of support systems such as wires or trellis systems for climbers and vines, which not only support plants but also enable the use of climbers without adhesive pads (Fig. 2). They create an air gap between a wall and vegetation, which protects a facade from severe weather conditions (Baran and Gultekin, 2017). Walls and facades directly covered by such plants are also categorised as green facades (Fig. 3) (Manso and Casto-Gomez, 2015). Plants can be planted directly into the soil near a wall or into pots.

In the case of their use on high-rise buildings, pots can be constructed at different altitudes (Fig. 2) (Dunnet and Kin-sbury, 2008).

Living walls are complex systems of plastic pots and modules or felt pockets (Fig. 4); they are always combined with irrigation and are secured on a metal frame, allowing for the use of nonclimbing species, grasses, shrubs, or succulents (Francis and Lorimer, 2011). The whole system is automatically controlled by a unit, which determines the dosage of fertilizer and water demands (Mir, 2011). We can state that green facades are simple and cost-effective solutions that do not require intensive care. However, they have a limited height; the selection of plants is narrowed only to climbers. The desired full-grown effect is achieved in years or decades. Living wall systems have a considerably higher initial cost but offer more planting options with various types of plants in different patterns. They do not have any height limitations, and the desired effect is achieved practically immediately. The plants are always watered and fertilised through an automatically operated irrigation system; hence they do not dry out and can provide a lush effect for decades. Constant irrigation and the freshness of the plants helps prevent a fire hazard, which is often associated with green walls. The differentiation overview by Mir is shown in Tab. 1 (Mir, 2011)

Evolving technologies can enable the use of different plant species. However, their selection should not only be based on aesthetic values. Essential factors in determining suitable species are the microclimate and the watering and sunlight requirements (Pérez-Urrestarazu, *et al.*, 2016). Plants selected for vertical gardens should be lightweight, with a shallow root system and ideally evergreen to keep the desirable effects in the winter, too. The selection of long-lived, rigid, and durable species decreases the costs of further replacement (Sarkar, 2018). Living walls offer the benefit of a wide selection of plants. Wildflowers, grasses and other plants that attract pollinators should be chosen to maximise the potential biodiversity of vertical gardens (Ivan *et al.*, 2022).



Fig. 2 Indirect green facade, Vienna (Zaťovičová, 2022)

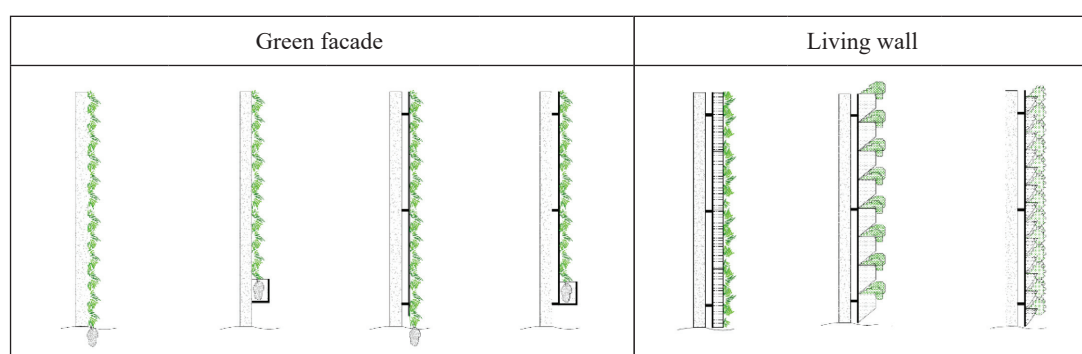


Fig. 3 Direct green facade, Vienna (Majorošová, 2012)



Fig. 4 Living wall, London (Majorošová, 2012)

Tab. 1 Types and Characteristics of Vertical Gardens (Mir, 2011)



Type	direct	direct	indirect	indirect	mat	pocket	plastic
Rooting in	ground	pot	ground	pot	textile	textile	soil
Support system	-	-	wire	wire	textile	pockets	modules
Plant type	climbers	climbers	climbers	climbers	up to shrubs	up to shrubs	up to shrubs
Air gap (mm)	0	0	50-3000	50-3000	>50	>50	>50
Vegetation thickness (mm)	200	200	100	100	≤400	≤350	≤450
Max height (m)	30	30	30	30	any	any	any
Weight (kg/m ²)	>5,5	>5,5	>54,3	>4,3	40-60	100	150
Irrigation	rain	system	rain	system	system	system	system
Full-grown in (y)	30	2-3	30	2-3	<1	<1	<1
Cost (€/m ²)	30-45	200	40-75	200	350-750	500-1000	400-600
Prefabric/In Situ	in situ	in situ/prefabric	in situ	in situ/prefabric	in situ	in situ	prefabric

3 RESULTS

The final design was created for a densely urbanised area of the historical city centre, which serves as a primary focal point for tourism and social life in Bratislava and is visited by many residents and tourists. We created a complex network of various vertical garden solutions on all the potentially suitable surfaces determined in the case study. Regarding the location of these potential spots on the main routes connecting important traffic hubs, such as bus and railway stations or public transport connection stops in the city centre, we proposed the idea of designing a navigation system using vertical gardens as direction points to the city centre to create an aesthetically appealing and ecological form of navigation system (Fig. 5). These points are frequently located at a convenient visual distance; hence, following them should be clear for longer routes. This also supports the findings by Vega and Küffer (2021) that small spots of greenery designed in a complex network could have a significant impact as an open green space. As determined in field analyses, three construction methods were examined based on different surface types, such as walls, which are typical for creating vertical gardens, less conventional column designs and free-standing green structures.

We are presenting three case study model designs herein, which could be applicable in various situations.

3.1 Vertical gardens on walls

In this case, a 9-story building on Dunajská Street was chosen. It is located in the middle of one of the city centre squares, and its architecture is incompatible with the surrounding buildings. This location has long been problematic since its architecture and infrastructure are deteriorating. However, it has great potential to become one of the main hubs for social life. The building selected has two sides wholly covered in advertisements (Fig. 6), so it is ideal for creating a green wall, making it aesthetically and environmentally beneficial (Fig. 5).



Fig. 5 Visualization of living wall (Zaťovičová, 2019)



Fig. 6 Current state of the facade with navigational arrow (Zaťovičová, 2019)

We have chosen a textile pocket system (Fig. 7), which is lightweight and simple to assemble. It gives plants enough space for rooting through and, thanks to multi-level irrigation, all the plants have an equal water supply. This technology can be used on facades of any scale. The construction has a 30 m height and 11 m width and is at a 130 mm distance from the façade, which prevents leakage from irrigation onto the wall; it further creates an air membrane that also serves as a thermal insulation layer. This type of vertical garden consists of a metal support frame mounted on a wall. Thick PVC hydrophobic foil is connected to the back of the frame. Water mixed with fertiliser is delivered to the plants via drip irrigation installed on the frame throughout the textile pocket

system. Excess water is collected by drainage at the foot of the wall and recirculated back into the irrigation system. The whole system is automated and controlled by a unit at the base of the construction.

The selection of suitable plant species will ensure that the idea of the navigation arrow will be clearly recognisable throughout the year (Fig. 8). *Sedum Spurium* 'Purpur winter' is hardy and does not require much care. It has red leaves that preserve the navigational function in the winter. At the same time, other species are designed to bloom gradually to attract pollinators from March to October, and always in the shape of a navigation arrow. *Hosta* 'Stifry', *Brunnera macrophylla* 'Alexander's great', and *Carex caryophyllea* 'The Beatles', were selected as filler species around the arrow. They have decorative leaves and inconspicuous flowers.

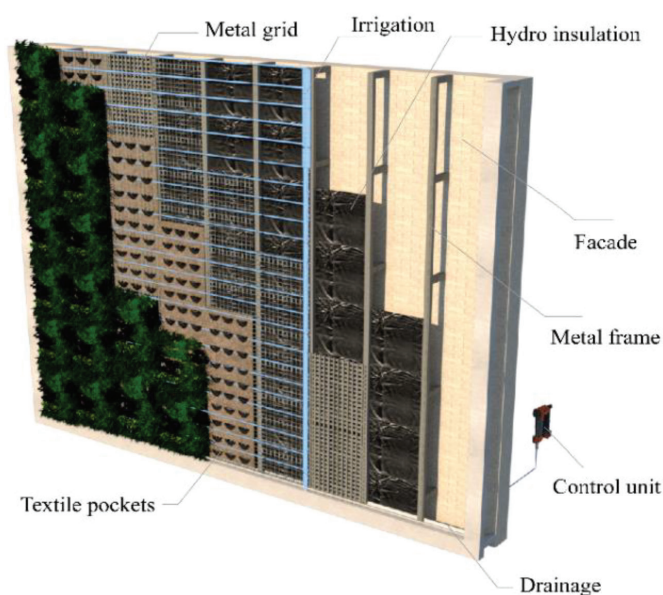


Fig. 7 Textile pocket system layers (Zaťovičová, 2019)



Fig. 8 Planting plan (Zaťovičová, 2019)



Fig. 9 Current state of the overpass (Zaťovičová, 2019)



Fig. 10 Visualization of the overpass (Zaťovičová, 2019)

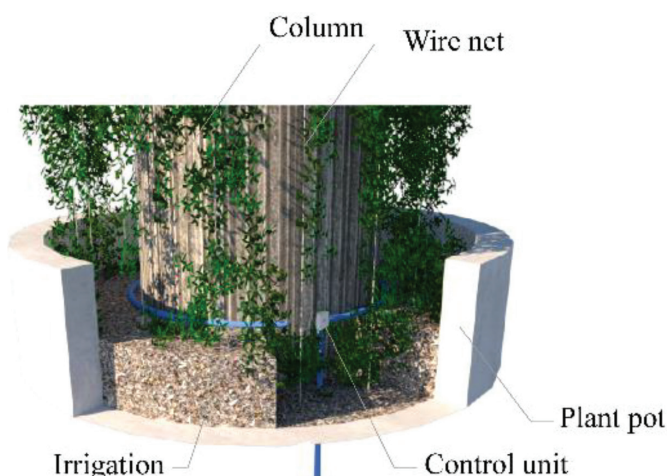


Fig. 11 Column structure layers (Zaťovičová, 2019)

3.2 Vertical gardens on columns

For this type of construction, the pedestrian overpass on Pražská Street was selected. It is a rigid concrete structure with no design value, and is located near the train station (Fig. 9). It connects several public transport stops, making it one of the first sights for visitors coming by train. The vertical garden offers a relatively quick and simple solution for increasing its aesthetic value and improving the area's microclimate (Fig. 10).

We created a simple construction (Fig. 11) consisting of a steel wire net mounted onto columns and pots located at the foot of the columns. The irrigation system is connected to the public water supply, thereby providing the required amount of water directly to the roots in the pot. The structure has a 7.2 m height, while the column perimeter is 800 mm. The designed pots have a height of 400 mm and a width of 300 mm, thereby providing enough space for a root system. We selected *Hedera helix*, which is hardy and tolerant of urban conditions. It grows relatively quickly, so it can provide the desired effect in a short time. Studies suggest that it can attract pollinators (Ivan *et al.*, 2022). The wire system provides support, mainly in the initial stages of growth. Over time, plants will be able to climb on horizontal parts of the overpass, thus enhancing its aesthetic and microclimatic value.

3.3 Free-standing vertical garden structure

Vertical gardens can also be made as free-standing structures when constructing them on more traditional surfaces is not available. They offer a less space-demanding alternative to conventional parks and give a free hand for creating various designs, shapes, and sizes. We chose a paved area on the Rázus embankment (Fig. 12) near a coffee house and former port of the Propeler boat. It is a Modernist architectural landmark of Bratislava created by the notable Slovak architect Emil Belluš. Despite its recent renovation and high-profile location on the city centre promenade, it does not attract many visitors. One of the reasons might be its large, paved surroundings without any greenery or shade, which suffers from significant overheating in the summer. Considering the historical importance of the area as a Propeler steamboat port, we designed a structure inspired by the shape of this boat (Figs. 13, 14, 16), which represents a symbolic gateway to the historical city centre from the other side of the river Danube (Fig. 15), as it used to be in the past. The metal structure comprises three columns at a 3 m distance apart on each side. Its lightweight design does not interfere with any architecture and views of the historical city centre and lets the vegetation stand out. Each side has a different height of the columns, i.e., 5.5 m and 7.5 m. The column located in the middle represents the steamboat chimney and is 12.5 m high. These vertical structures are connected with steel wire, enabling climber plants to grow horizontally, too. This will, over time, create the desired shade. There are pots at the foot of each column with dimensions of 110 x 110 cm; their height will allow people to sit on their edges. The whole structure has a 10 m width and a 20 m length. Fast-growing *Hedera helix* will achieve the desired effect in a relatively short time. The idea of navigating to the city centre will be preserved by *Parthenocissus quinquefolia*, which will grow on a horizontal wire arrow.

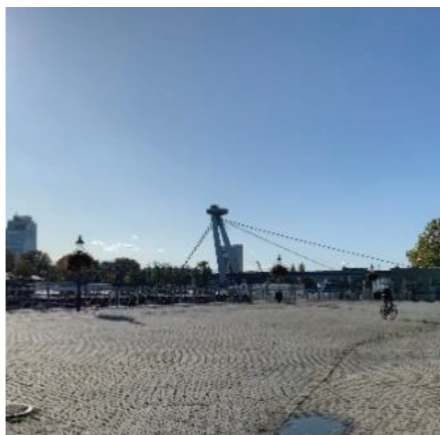


Fig. 12 Current state of the location
(Zaťovičová, 2019)

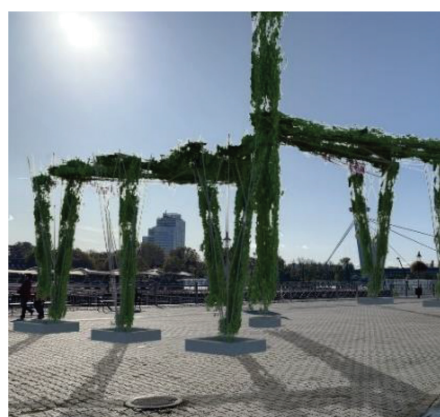


Fig. 13 Visualization (Zaťovičová, 2019)



Fig. 14 The first Propeler steamboat
(<https://sk.pinterest.com/pin/537124693037095777/>)



Fig. 15 Aerial view (Zaťovičová, 2019)



Fig. 16 Visualisation of the structure (Zaťovičová, 2019)

4 CONCLUSION

Intensive urban development and global warming are rapidly becoming major issues, and it is thus becoming more necessary to emphasise the importance of urban greenery. This article proposes various options for vertical garden creations in different situations. Vertical gardens offer a broad palette of construction methods for different surfaces at various price ranges. They present an alternative solution for supporting biodiversity and mitigation of the overheating of such urban areas where that is not possible by creating new horizontal green areas. They can transform these measures to vertical dimensions while effectively utilising bare surfaces prone to overheating that would otherwise be unused. They do not take up valuable land space and can be relatively quickly and easily constructed on existing structures, thereby offering positive microclimatic, economic, and aesthetical values.

This article proposes a practical application of green structures with additional design value using the example of the Old Town district of Bratislava. We examined all the potentially suitable surfaces and created a network of the maximal usage of these spaces by proposing a complex and systematic look at the absence of vegetation in the city centre. We have also presented the idea of using these structures as a navigational system since they are often located on important routes connecting transport hubs with the city centre. Specific types of constructions on facades, columns, and free-standing structures in the most exposed parts of the area were described in the case study examples.

Vertical gardens offer plenty of benefits regarding the microclimate and positive effects on the human psyche and energy efficiency of buildings. Their implementation in Bratislava's urban environment and globally is still relatively rare, making them worthwhile sights for visitors and residents. However, it is essential to consider these structures not only as aesthetic elements but also as relatively simple solutions to mitigate the overheating of facades and bring natural elements to areas where the surroundings do not allow for a conventional approach.

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