

ANALYSIS OF INDOOR AIR QUALITY IN A KINDERGARTEN

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Abstract

Today in addition to the design of structures, layout solutions, and the design of suitable materials, the modern construction industry also addresses meeting the requirements for the energy performance of a building, with which the topic of the quality of the indoor environment is fully intertwined. Comfort in the use of buildings, and especially the provision of thermal comfort, is a fundamental aspect in the design of technical equipment systems, where a properly selected system regulating the indoor environment can affect both the energy performance of the building and the quality of the indoor environment. One of the important factors is the air quality, where the main factor that affects the indoor environment is the concentration of CO₂, whose value affects the biological functions of the human organism. The subject of this research is an evaluation of the indoor air quality in a kindergarten because children are more sensitive to environmental influences.

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Key words

- Indoor environment of a building,
- Kindergarten,
- Thermal comfort,
- Carbon dioxide,
- Ventilation,
- Heat demand.

1 INTRODUCTION

People spend most of their time, i.e., up to 90 %, in buildings. Therefore, the quality of the indoor environment in buildings should be one of the most important parameters in building design. Therefore, when implementing measures to reduce the energy performance of buildings and when building new buildings, the quality of the indoor environment should not be neglected in addition to the energy performance requirements. Alongside health care facilities, buildings for the education of children and young people are the most monitored buildings in this regard. Therefore, in these buildings, compliance with CO₂ limits is stressed (Bogdanovica et al., 2020), of course, together with other monitored variables such as the indoor air temperature and relative humidity (Coggins et al., 2022). The issue of indoor environmental quality is mainly addressed in primary school buildings, where air quality measurements are carried out within the

framework of professional publications (Skotnicova et al., 2019) (Talukdar et al., 2020), including proposed measures leading to their improvement. However, few measures are devoted to kindergarten buildings, where it is also important to monitor compliance with the relevant parameters, given the sensitive organisms of preschool children.

2 INDOOR ENVIRONMENTAL QUALITY

An optimal indoor microclimate is a prerequisite for the well-being and health of its inhabitants. It is important to note that the concept of an optimal microclimate can be a very subjective assessment, unless we talk about it only in terms of standard requirements (Coggins et al., 2022). Thermal comfort is closely related to an optimal microclimate. Thermal comfort in an indoor environment is one of the fundamental factors that influence the quality of an indoor environment and make

a significant contribution to the satisfaction of the occupants (Talukdar et al., 2020). Thermal comfort means that thermal conditions are achieved where a person is neither too cold nor too warm and therefore feels comfortable. A person feels best in a climate range of 20 to 22°C and 40 to 60% relative humidity. Climatic conditions outside this range are perceived by most people as uncomfortable (McGill et al., 2015). The temperature and humidity are essential factors that influence the optimal microclimate. Both variables are influenced by the parameters of the external environment and can be controlled in a targeted manner. The basic prerequisite is the correct design of the thermal engineering properties of the building structures that enclose the interior space of the building under consideration and form the envelope of the building. Compliance with the building envelope level is part of the requirements for the energy performance of buildings.

In terms of air temperature, it is also necessary that the heating system is designed to achieve the design indoor temperatures in individual rooms in the winter period, at the same time, effective regulation is necessary to avoid overheating. In the summer, there should be no unwanted overheating of the interior spaces (Cihelka, 1976).

In addition to the standard requirements for the temperature, humidity, and ventilation, one of the main indicators of air quality is the concentration of carbon dioxide (CO_2), whose value informs the quality of the ventilation of the indoor space. It is expressed in ppm units (parts-per-million). The ppm units means the number of volumetric units of CO_2 in one million volumetric units of air. In an outdoor environment, the CO_2 concentration is approximately 0.04 %, that is, 400 ppm. In terms of hygiene standards, the Czech Decree No. 20/2012 Coll., provides as follows: “Residential rooms must have sufficient natural or mechanical ventilation and be adequately heated with the possibility of regulating the internal temperature. For ventilation of living rooms, a minimum amount of outdoor air exchange of 25 m³/h per person or a minimum ventilation rate of 0.5 l/h shall be ensured during the stay of the persons. Carbon dioxide CO_2 shall be used as an indicator of the quality of the indoor environment and shall not exceed a concentration

of 1500 ppm in the indoor air.” In addition to CO_2 , indoor air can also contain other pollutants such as formaldehyde, carbon monoxide, VOCs, radon, asbestos, dust particles, odours, and microorganisms. Proper ventilation of the building will, of course, ensure that the concentration of these substances is also reduced.

The requirements for ventilation and microclimatic condition parameters that apply to the premises and operation of facilities and establishments for the education and training of children and adolescents are specified in the Czech Decree No. 410/2005 Coll. In the current version of the Czech Decree No. 410/2005 Coll. dated 22th October 2022, the amount of the fresh air supply in classrooms has been reduced from 30 m³/h per pupil to 20 m³/h per pupil.

3 EXPERIMENTAL MEASUREMENT

3.1 Subject of the measurement

The subject of the measurement is a single-storey kindergarten building. The floor plan is L-shaped with dimensions of approximately 40 x 20 m, and the built-up area is 598 m². The building lacks a basement and has a flat roof. It has been in operation since 2013. In terms of layout, the kindergarten is divided into three separate units consisting of three departments according to the age of the children and the related facilities for these departments. The total capacity of the kindergarten is 60 children. Each department has a separate entrance from the exterior. A separate terrace is adjacent to the day rooms of each department, which is accessible through French windows. The heating of the building is handled using a block heat transfer station located in the technical room of the building, which is connected to the central heating system. The heating surfaces in the building are panel heating and underfloor heating. There is an air handling unit installed in the building, which provides ventilation for areas without any natural ventilation through windows. The children’s day rooms are ventilated by windows that open.

Tab. 1 Effect of CO_2 concentration on the human body

Concentration CO_2 (ppm)	Evaluation and effect of CO_2 concentration
350-500	normal outdoor concentration
500 - 800	high level of healthy indoor environment
800 – 1,000	acceptable level of healthy indoor environment
1,000	recommended value
1,000 – 1,500	complaints of odours, feeling of heavy (foul) air, slight fatigue
1,500	limit value – Decree no. 268/2009 Coll.
1,500 – 2,500	decrease in concentration, fatigue
2,500 – 5,000	dullness, fatigue, possible health problems
>5,000	nausea, increased heart rate
>15,000	breathing difficulties
>40,000	possible loss of consciousness

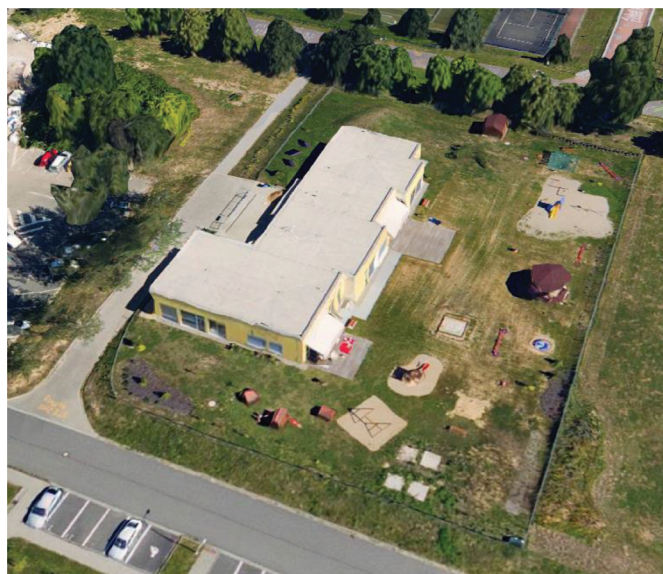


Fig. 1 Aerial photo of the kindergarten

3.2 Method

A classroom attended by 24 children aged 5 to 6 years was selected for the measurements. Class occupancy is from Mondays to Fridays from 7:30 am to 5:00 pm. It is a rectangular room with dimensions of 19 x 5.8 m and a floor area of 109 m². The room has three outer walls, the longest of which is oriented towards the southwest. The ventilation of the classroom is provided naturally by opening the windows. The windows are fitted with external blinds. For ventilation in the summer and also in the winter, the possibility of fully opening the French window leading to the terrace is used. Heat and humidity sensors and CO₂ sensors were placed in the classroom. Given the shape of the room and the division of the room into two halves, i.e. a dining and working area with tables, and a play and re-

laxation area, two measuring points were created to measure the necessary quantities as realistically as possible (see Fig. 2).

The values were recorded every 15 minutes on data loggers. The data logger is an appliance that allows the connection of up to four measurement inputs. It is equipped with an LCD display and a keyboard. The display shows the current values of the measured parameters, and it is also possible to enable storage of the data into the memory of the data logger. Sensors and a data logger from Ahlborn were used for the measurements.

Apart from the actual measurements, the number of children present and the activities performed within the schedule were recorded on a daily basis using the sensors, e.g., resting activity (drawing, independent play, etc.), physical activity (exercise, movement games, etc.), and being in the garden (without the presence of children in the measured area). In addition, the frequency of the ventilation of the space was also recorded. For completeness, the outdoor environment parameters were measured using a professional weather station installed on the roof of the building. Measurements were taken every week from Monday to Sunday in the summer period (June), the transition period (October), and the winter period (November), when the heating season had already started, and expected cooling had occurred.

3.3 Results

Through the experimental measurements, it was found that the CO₂ concentrations in June, i.e., in the summer, were in the range of 350-500 ppm, i.e., they reached the same level as the CO₂ concentration in the outdoor environment. The measurements carried out in the months of October and November indicated that CO₂ concentrations in the area addressed were in a range of 400-1400 ppm. Fig. 3 shows the course of the CO₂ concentration for a week in the month of October. It is obvious that there are significant fluctuations in values, which are caused by the diverse physical activity of the children and

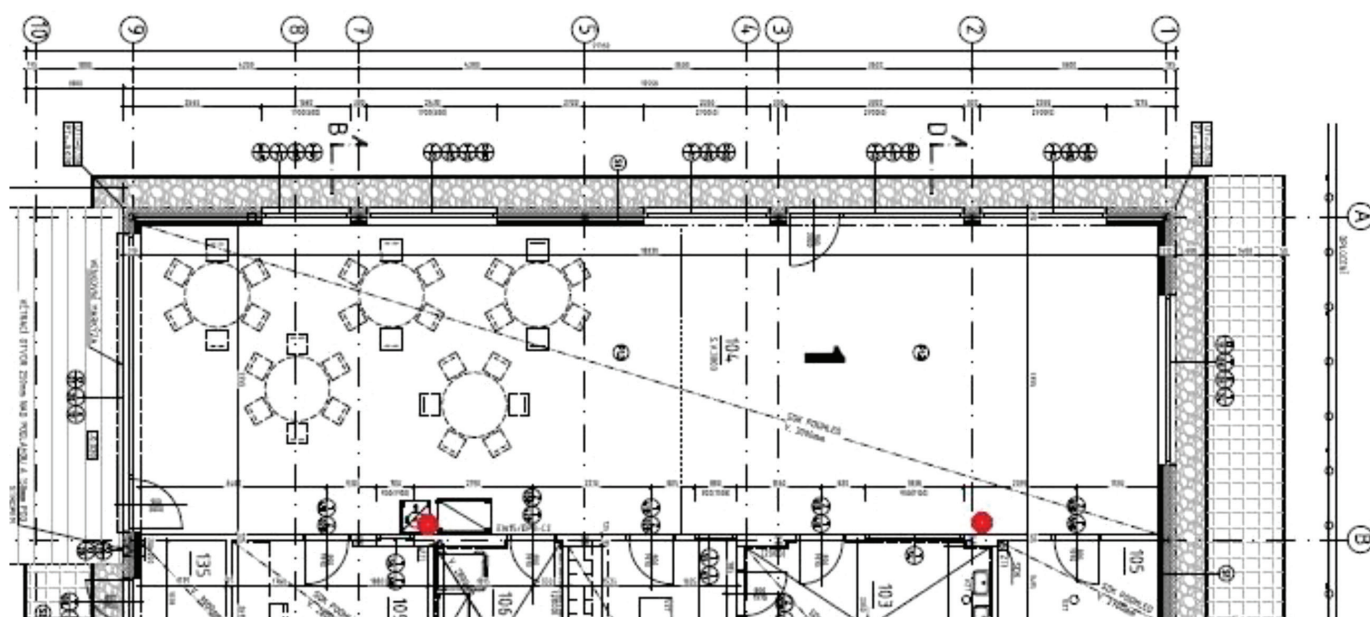


Fig. 2 Layout of the classroom

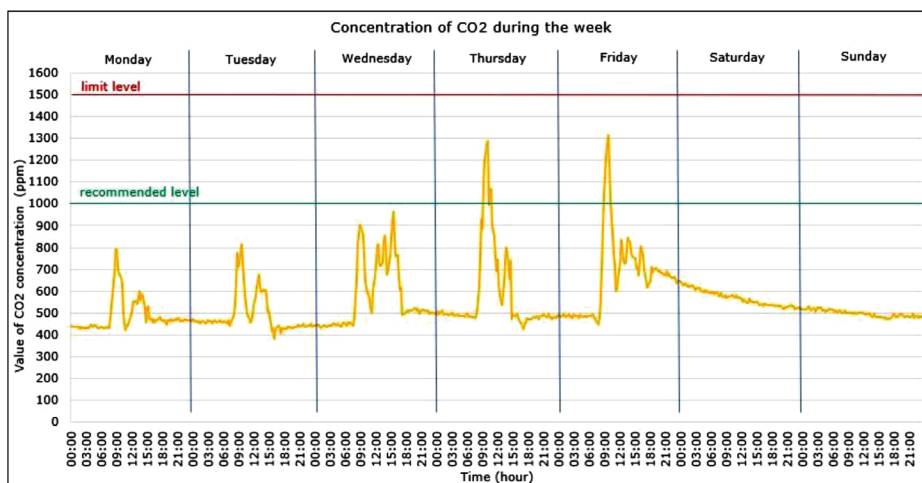


Fig. 3 Graph of the CO_2 concentration during the week (October)

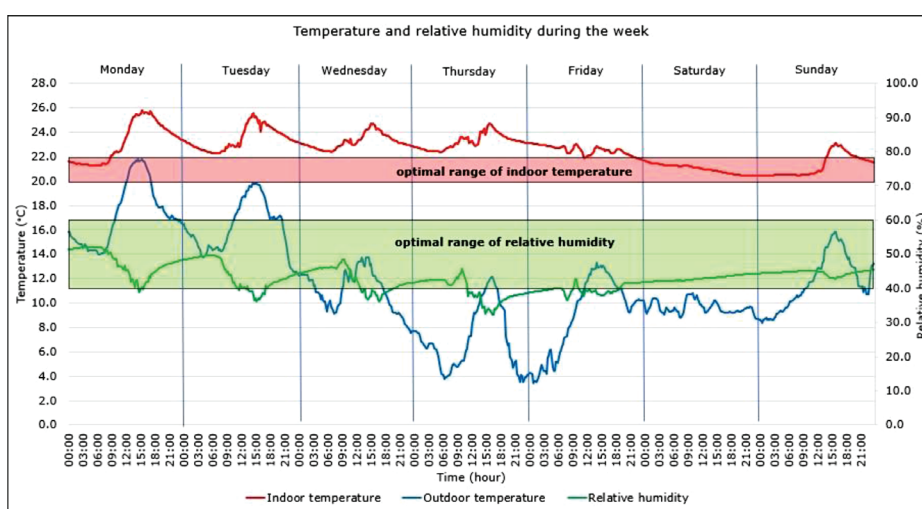


Fig. 4 Graph of the temperature and humidity during the week (October)

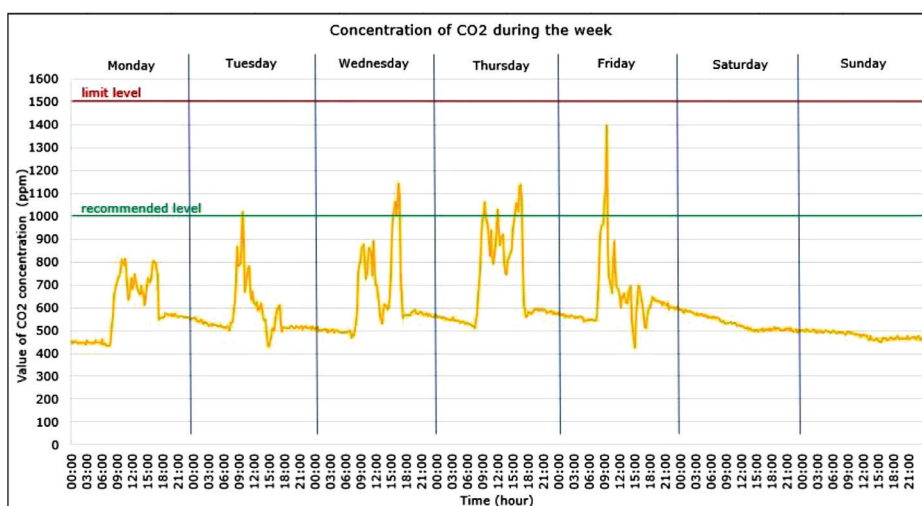


Fig. 5 Graph of the CO_2 concentration during the week (November)

their presence in the classroom or in the garden. As expected, the highest concentration values were mainly in the morning during physical activity.

Fig. 4 shows the course of the internal temperature and relative air humidity during the period evaluated. It is evident that

the values measured of the internal temperature are outside the optimal range from the point of view of the assessment of thermal comfort. This graph also records the course of the outdoor air temperature, which affects the indoor air temperature and the frequency of ventilation.

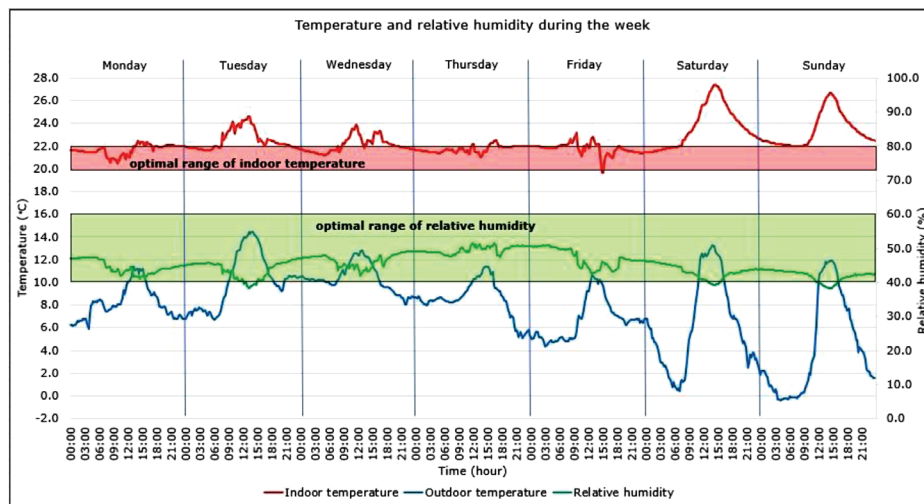


Fig. 6 Graph of the temperature and humidity during the week (November)

The activity record during the measurement shows that ventilation was provided throughout the day by opening two hinged windows. In the case of higher outdoor temperatures, the doors to the terrace were also opened. Fig. 4 shows that on Thursday and Friday, the outdoor temperatures were lower. At the same time, it follows from the activity records on these days that the frequency of ventilation through the door to the terrace was not the same as on other days in this period. Therefore, the measured concentrations exceeded the 1000 ppm limit, which is not a limit value, but a recommended value.

From the point of view of the marginal conditions of the measurements, the month of October 2022 was above average in terms of temperature; as shown in Fig. 4, temperatures reached up to 20 °C at the beginning of the week measured. The experimental measurements were repeated in the month of November, when expected cooling had already occurred and the supply of heat to the building under consideration had begun. It can be seen in Fig. 5 that the measured values of CO₂ were higher than the previous measurement and that the recommended level of the CO₂ concentration was exceeded 4 times. This was due to a decrease in the frequency of ventilation of the space due to lower outdoor temperatures, as is clear from Fig. 6.

4 ENSURING INDOOR AIR QUALITY

One of the main indicators of air quality is the concentration of carbon dioxide (CO₂), the value of which indicates the quality of the indoor ventilation (Wallace et al., 2002). The most common and simplest method of ventilation is natural ventilation, in which the indoor air with a higher carbon dioxide concentration is exchanged for outdoor air with a lower carbon dioxide concentration. However, this may not be the case in locations that are affected by increased traffic in the vicinity. In the past, infiltration through leaks in window joints was sufficient to provide the necessary room air exchange, with an exchange rate of 2 to 4 times the room's air per hour (Cardoso et al., 2020). Today, when new buildings are constructed or existing window openings are replaced with new high-leakage window openings, the air exchange rate drops below 0.5 times the room volume per hour. It is also important

to note that with natural ventilation through window openings, it is not possible to control the air exchange rate according to the actual state of the indoor environment. In this case, the frequency and length of the ventilation is completely dependent on the person who can quickly acclimatize in the environment. Acclimatisation refers to the process by which occupants who have been in a room for some time become less sensitive to contamination of the indoor microclimate by odours, so that the unsatisfactory quality of the indoor environment is always best assessed by a newly arrived person. Therefore, providing sufficient ventilation through the windows is problematic unless the space is fitted with sensors to measure the indoor environmental quality.

If a room is designed to be naturally ventilated through windows, the room should be equipped with sensors to measure indoor air quality, that is, sensors for the concentration of CO₂ or other pollutants such as volatile organic compounds (VOC) (Ahn et al., 2022). Temperature and relative humidity sensors are also suitable for monitoring thermal comfort. Based on the CO₂ concentration value and when a set limit is exceeded, the measuring instrument will be signalled, e.g., by illuminating a light, and the windows should be kept open for a certain period of time, e.g., 10 minutes. In the case of a building with intelligent control, the windows (or vents) will automatically open, and the windows will close when the CO₂ concentration falls below the set level. However, the disadvantage of natural ventilation is that heat is removed along with exhaust air in the winter. The heat loss through this ventilation must be covered by the heating system, resulting in an increase in the heating demands and the design of a heat source with a higher rated capacity. In addition, natural ventilation can cause thermal discomfort due to cold air entering the room in the winter.

The second way to maintain the quality of an indoor environment is to design a forced ventilation system. Ventilation in this case can be designed for a constant air flow rate, whose value is set by legislation or, even better, based on the measured CO₂ concentration using sensors. Forced ventilation operates automatically for the duration of a building's operation according to the requirements set and, where appropriate, a time schedule. The forced ventilation system should be supplemented by the installation of heat recovery equipment.

This transfers the heat from the exhaust air to the fresh air supplied to the interior. In this case, the energy consumption of the ventilation is reduced. Forced ventilation can be designed to be central for a whole building or local for a room or group of rooms. A local ventilation system is particularly suitable for existing buildings, e.g., typically for classrooms in school buildings, where a separate ventilation unit is designed for each classroom located directly in the space to be designed.

A comparison of the different ventilation methods in terms of energy demands and ventilation costs can be shown by the example of the class that was the subject of the experimental measurements described in Section 3. The air volume of the classroom is 327 m³. The capacity of the classroom is for 24 children. According to current legislation in the Czech Republic (Decree No. 20/2012 Coll.), the amount of the air supply should be 20 m³/h per pupil. Therefore, the total amount of the air supply required for the classroom is 520 m³/h. If we want to evaluate the energy consumption of the ventilation, it is necessary to consider the heat demands for ventilation throughout the heating period. On site, the design outdoor air temperature in winter is -15 °C; the average outdoor air temperature in the heating period is 4 °C; and the number of heating days is 229. Therefore, for the heating period there are 1554 hours of operation of the building, corresponding to the number of the operating hours of the ventilation and a total volume of 808,080 m³ of the fresh air supply.

In order to determine the energy demands, it is necessary to know the heat loss through ventilation. According to CSN EN 12831-1 (2018) and CSN EN ISO 52016-1 (2019), the design ventilation heat loss for a heated space is calculated by the following equation:

$$\Phi_{V,i} = H_{V,i} \times (\theta_{int,i} - \theta_e) \quad (\text{W}) \quad (1)$$

where $H_{V,i}$ is the design ventilation loss factor (W/K); $\theta_{int,i}$ is the design indoor temperature in the heated space (°C); and θ_e is the design outdoor temperature (°C). The design ventilation loss factor is expressed by the mathematical formula below:

$$H_{V,i} = V_i \times \rho \times c_p \quad (\text{W/K}) \quad (2)$$

where V_i is the air exchange in the heated space (m³/h); ρ is the air density by $\theta_{int,i}$ (kg/m³); and c_p is the specific heat capacity (Wh/kg.K). When natural ventilation is designed in the heating space, the value of the air exchange in this space is at a maximum, i.e., the air exchange by infiltration or the minimum air exchange required for hygiene reasons. For a structure with a very tight building envelope, the air exchange value required for hygienic reasons, which is given in the Czech Decree No. 410/2005 Coll., is considered. Based on the mathematical relationships mentioned above (1)(2), the heat loss through the ventilation of the design for the class under consideration is 6.05 kW.

Calculating the heat demands for heating and ventilation can be performed in several ways with different degrees of accuracy, and the value of the heat demands for heating can vary depending on the method used and the parameters chosen (CSN EN ISO 52016-1, 2019). The choice of specific parameters and the calculation method also depends on the purpose of the calculations. At the conceptual design stage, without any knowledge of the detailed building design, the grad-day method

can be used, which is based on a single-zone quasi-stationary model with an annual time step. The grad-day method is based on the heat loss of the building and the climatic data of the building site and considers the factors of limited night-time operation, building accumulation, and the noncontemporaneity of the heat loss by transmission and infiltration. If the heat loss through natural ventilation is covered by the heat source for heating, then the annual heat demand is calculated using this method according to the following equation:

$$Q_{H,a} = \frac{24 \times \Phi_{HL} \times \varepsilon \times D}{\theta_{is} - \theta_e} \quad (\text{kWh/a}) \quad (3)$$

where Φ_{HL} is the heat loss of the structure (kW); ε is the correction factor for the temperature reduction, shortening the heating time, and non-simultaneity of heat loss through ventilation and heat transfer (-); D is the number of grad-days (K.day); θ_{is} is the average calculated internal temperature (°C); and θ_e is the external temperature calculated (°C). If only the heat demand for natural ventilation is calculated, then instead of the structure's heat loss Φ_{HL} , the value of the heat loss through ventilation is substituted for by $\Phi_{V,i}$. A crucial parameter that significantly affects the resulting value of the energy demands for ventilation is the number of grad-days. Unlike heating, which mostly operates continuously during the heating period, depending on the type of building operation, the ventilation system may only operate for a few hours a day, depending on the presence of the occupants. Therefore, it is important to correctly determine the number of operating hours of the ventilation system during the heating period. According to relation (3), the heat demand for ventilation for the class is 3961 kWh/a, i.e., 14.3 GJ/a.

In the Czech Republic, the cost per 1 GJ of heat to cover the heating needs of the building in 2021 was € 18. With the current increase in energy prices, the cost may increase to € 83 per 1 GJ in 2023. On this basis, the cost to cover the annual heating needs for ventilation may be up to € 1,187 in 2023. If a forced ventilation system with a heat recovery efficiency of 60 % (minimum seasonal heat recovery efficiency) is installed in the building, the cost of the annual heat demand for ventilation would be € 475. This means an annual savings of up to € 712. The purchase cost of an air handling unit with heat recovery for the class to be addressed ranges from € 4,149 to € 7,469. The simple return on the investment is 6-11 years. If we were to consider the realistically achievable efficiency of the heat recovery to be around 80%, then it is possible to achieve a payback of up to 4 years.

In this context, it should also be mentioned that, in addition to the proven energy savings in ventilation, a forced ventilation system is comfortable and requires minimal maintenance, which means regular replacement of the air filters, the frequency of which depends on the level of outdoor pollution and the type of operation of the building. This is also related to a possible need to clean the air ducts.

5 DISCUSSION

Our research shows that the limit value of the CO₂ concentration, which is the main indicator in the assessment of

indoor air quality, is not exceeded in the class evaluated. The measured values exceed the recommended concentration level only exceptionally. However, it should be mentioned that although the measurements took place during the heating season, they were performed during the period when the outdoor temperatures had not yet fallen below freezing. From the point of view of thermal comfort, it could be said that the internal temperature of the interior is outside the optimal range. This can be caused by the improper regulation of the heating system or an inappropriate overheating of the space being treated above the design value of the indoor air temperature given by the relevant regulations. As already mentioned, the assessment of thermal comfort is very subjective. However, it was found that precisely based on higher values of internal air temperatures and the feeling of the air exhaled, targeted ventilation of a window takes place, which subsequently contributes to reducing the concentration of CO₂ and ensuring the good air quality of the internal environment. For this reason, the measured CO₂ values are above any low expectation. Natural ventilation is uneconomical from the point of view of an energy building in the winter and leads to an increase in the heat demand for heating.

We must not forget, however, the effect of natural ventilation on the human mind. Natural ventilation is a daily repetitive activity in which our sensory experiences, smells coming from outside, the feeling of comfort, and the pleasure of a breeze play a major role. On the other hand, we remove odours from the interior that are unpleasant to us. According to (Hauge, 2012), it is known that natural ventilation is a part of practical, aesthetic, and social needs, in the sense of caring for other occupants of a building. Furthermore, natural ventilation is especially important in the so-called transition period when people adapt to a change of season. Natural ventilation in this period induces

a feeling of freedom. Moreover, in the transition period, the outdoor air temperatures are high enough, and there is no longer a significant increase in the building's energy efficiency. If we would like to combine the advantages of both systems, we can recommend the use of forced ventilation with heat recovery in the winter and natural ventilation in the interim period, which could support the good mental state of the occupants.

6 CONCLUSION

The purpose of this article was to point out the importance of monitoring and maintaining the quality of the indoor environment in buildings, as it has a direct impact on human health. The basis for the evaluation of the internal environment of the selected kindergarten was the measured values, which did not significantly exceed the recommended values. However, they demonstrated how important ventilation is to ensure high indoor air quality. Furthermore, the article proved that by using forced ventilation with heat recovery, it is possible to reduce the energy demand of the building, especially within the framework of the need of ventilation for heat.

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VERIFICATION OF THE RAMBERG-OSGOOD MATERIAL MODEL FOR THE FIRE DESIGN OF STEEL MEMBERS

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Abstract

In this paper, the modified Ramberg-Osgood constitutive equations for calculating the stress-strain of steel at elevated temperatures using the parameters determined based on the transient state tensile test results achieved at the Helsinki University of Technology are verified. This is done by numerically comparing the global and local buckling capacities of I-shaped steel members incorporating the modified Ramberg-Osgood model along with the material model given in the fire section of Eurocode EN1993-1-2. For this purpose, a numerical model using the ABAQUS software was developed. Then, nonlinear analyses with imperfections (GMNIA) were performed to compare the buckling capacities of the steel columns and beams of four different hot-rolled cross-sections (IPE160, IPE180, HE100B, HE500B), made of steel grade S355, at three different temperatures (400°C, 500°C, 600°C). The results showed that adopting the modified Ramberg-Osgood model can lead to the same buckling capacities resulting when using the EN1993-1-2 material model for steel temperatures of less than 400°C. However, adopting this model for 600°C overestimates the buckling capacities in most cases.

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Key words

- Ramberg-Osgood,
- Fire design,
- Structural steel,
- Material model,
- Elevated temperatures,
- Nonlinear analysis.

1 INTRODUCTION

1.1 Literature study

Numerical modeling is considered nowadays to be an effective tool for investigating and proposing new design methods. An extremely important step to build a reliable numerical model, which is able to predict the actual buckling capacity of steel members at elevated temperatures, is the correct application of the accurate mechanical properties of the material/materials used. It is a well-known fact that when the temperature increases, the mechanical properties of steel, especially its strength and stiffness, reduce dramatically, and this considerably affects the structural response of steel members and may lead to structural instability in steel structures as a whole.

In the literature available, the material models of steel at elevated temperatures are usually given in the form of stress-strain curves. The mechanical properties of steel (e.g., the modulus of elasticity, yield stress and ultimate strength) can be obtained from these curves. These data are obtained from two types of tensile tests of steel at elevated temperatures, namely, steady state and transient state tests. In steady-state tests, the specimen is first heated to the required temperature before straining to failure. The specimen in transient-state tests is first subjected to the load, and then the temperature is increased until failure. Kirby and Preston (1988) reported that transient-state tests give lower values of strengths than the values obtained from steady-state tests on carbon steel for a temperature range of (400 – 800) °C. There are many material models available in the literature. For a practical design, EN 1993-1-2 (2005) adopts Rubert and Schau-

mann's model, which is derived from transient-state tests on beams (Rubert and Schaumann, 1986). In addition to the deterioration of strength and stiffness when the temperature increases, the clear yield point at a normal temperature disappears at elevated temperatures, and the yield strength becomes difficult to identify as the behavior becomes highly nonlinear with the rising temperature. For this reason, and because of large strains shown in steel members at elevated temperatures, it is more usual to consider the effective yield stress at a total strain within a range between 0.2% to 2.0% (ECCS, 1989; BS 5950, 2000). EN 1993-1-2 adopts an effective yield strength at a 2.0% total strain for classes 1,2,3 cross-sections. For class 4 cross-sections, the 0.2% proof strength should be used (EN 1993-1-2, 2005).

The material model investigated in this paper is that given by (Outinen et al., 1997) it was proposed using the calculating method developed by (Ramberg and Osgood, 1943); the mechanical properties of S355 steel at elevated temperatures are calculated using simple formulas based on transient state tensile test results carried out at the Laboratory of Steel Structures at the Helsinki University of Technology (SFS-EN 10 002-5, 1992; EN 10 002-2, 1992; EN 10 002-4, 1992). These formulae are proposed instead of the reduction factors given in EN1993-1-2. Wang et al. (2012) reported that the Ramberg-Osgood equations are capable of providing a convenient way to express stress-strain curves as a continuous function. Very important research was conducted by (Pauli et al., 2012), who carried out extensive experimental investigations on the material behavior of carbon steel at elevated temperatures and on structural stub and slender columns on fire. It was reported that the EN1993-1-2 material model for carbon steel has difficulties in describing the stress-strain relationships from tensile tests due to the fact that it overestimates the strain hardening for strains smaller than 2% and underestimates it for larger strains. Moreover, the shape of the modelled stress-strain curve of EN1993-1-2 cannot be adapted to the individual stress-strain relationships of the experimental results. On the other hand, adapting the one-stage Ramberg-Osgood model and its modification by Gardner-Nethercot [Gardner and Nethercot, 2004], has led to the better modelling of experimentally obtained individual stress-strain relationships of different steel grades and temperatures. Moreover, for high temperatures above 600 °C, it has been shown that the stress-strain relationship has an almost bilinear shape (Pauli, 2012). Another disadvantage of the Eurocode material model was reported by (Wang et al., 2012) who stated that the rate of change of the gradients (the tangent modulus) at the points between the limit of proportionality $f_{p,\theta}$ and yield points $f_{y,\theta}$ is not continuous, even though the values of the stress and tangent modulus are continuous. According to (Knobloch et al., 2010), the effect of the nonlinear stress-strain relationship of steel at elevated temperatures on the overall buckling strength is of high importance. In addition, it was concluded that adopting the temperature-dependent stress reached at a 2 % strain leads to unreliable results for the cross-sectional capacity in pure compression. Moreover, the unsafe nature of the EN1993-1-2 curve was also reported by (Nemer and Papp, 2021).

1.2 The aim of the current research

Thorough knowledge and proper implementation of the behavior of materials at elevated temperatures is extremely important for developing accurate numerical models, which

are able to predict the real behavior of steel members at elevated temperatures. Additionally, improving the accuracy of the material models incorporated in a fire design is also important with regard to the safety and economy of a steel structural design. Based on the available literature given above, it can be stated that there is a lack of certainty in and many disadvantages of the current EN1993-1-2 material model. Therefore, this paper aims at evaluating the applicability of the modified Ramberg-Osgood material model as an alternative material model for the fire design of steel members. The parameters for this fire design model were presented by (Outinen et al., 1997) based on an actual test that was not investigated or verified later by other researchers.

1.3 The content of the paper

In this paper, the EN1993-1-2 and the modified Ramberg-Osgood material models are first presented; then the numerical model is explained. Subsequently, the local buckling capacity of a steel beam studied by (Prachar et al., 2016) and the global buckling strengths of columns and beams are calculated by a geometrically and materially nonlinear analysis (GMNIA) incorporating the two material models.

2 MATERIAL MODELS AT ELEVATED TEMPERATURES

This section covers the description of two material models, which represent the material response of carbon steel at elevated temperatures, i.e., (1) the EN1993-1-2 material model and (2) the Ramberg-Osgood material model.

2.1 The EN1993-1-2 material model

The material model for carbon steel at elevated temperatures, as given in the fire section of Eurocode (EN1993-1-2, 2005), is presented in Fig. 1. The response of the material is divided into four stages as follows:

The first stage is linear-elastic up to the proportional limit

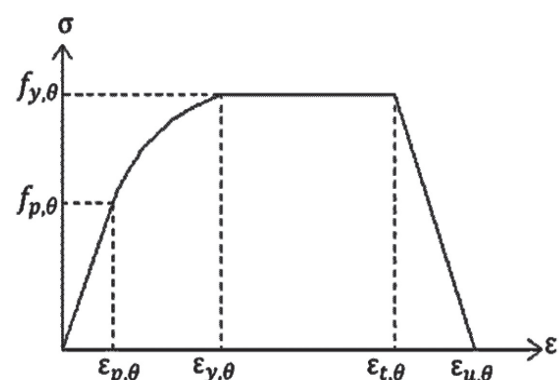


Fig. 1 EN1993-1-2 material model for steel at elevated temperature θ [1]

point $f_{p,\theta}$ (the end of the elastic stage), at which the stress is proportional to the strain. At this stage, only two basic material parameters are needed, i.e., the slope of the linear elastic range

for steel at elevated temperatures (E_θ) and also the proportional limit for steel at elevated temperatures ($f_{p,\theta}$). Then, the linear elastic range is followed by an elliptical curve between the proportional limit $f_{p,\theta}$ to the effective yield strength of steel at an elevated temperature ($f_{y,\theta}$), which is defined as the strength at a 2% total strain. In the third stage, the stress remains constant between $\varepsilon_{y,\theta} = 2\%$ and the limiting strain for the yield strength, $\varepsilon_{t,\theta} = 15\%$. In the last stage the stress drops to zero at the ultimate strain $\varepsilon_{u,\theta} = 20\%$, which means a fracture.

Fig. 2 shows the stress-strain curve according to the fire section of Eurocode EN1993-1-2 for S355 steel at three different temperatures (400, 500, 600 °C).

In EN1993-1-2, the strength and stiffness degradation of

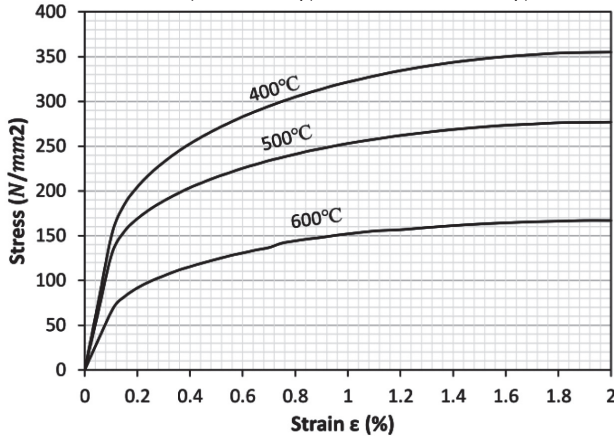


Fig. 2 EN1993-1-2 stress strain curve at elevated temperatures for S355

steel at an elevated temperature is presented using reduction factors, where the property at an elevated temperature is normalized with respect to the equivalent property at an ambient temperature. Table 1 presents the reduction factors for the effective yield strength ($k_{y,\theta} = f_{y,\theta}/f_y$), proportional limit ($k_{p,\theta} = f_{p,\theta}/f_y$) and elastic modulus ($k_\theta = E_\theta/E$) at the three temperatures (400, 500, 600 °C) investigated, where f_y , E are the yield strength and the modulus of elasticity of the normal temperature design, respectively.

Tab. 1 Reduction factors at steel temperature θ as given in EN1993-1-2

Temperature			
400 °C	1.00	0.42	0.70
500 °C	0.78	0.36	0.60
600 °C	0.47	0.18	0.31

2.2 The Modified Ramberg-Osgood model based on a real transient test

The equation for calculating the stress-strain curve of steel at elevated temperatures proposed by (Ramberg and Osgood, 1943), is the following:

$$\varepsilon_i = \frac{\sigma_i}{E_{a,\theta}} + \beta \cdot \left(\frac{f_{y,\theta}}{E_{a,\theta}} \right) \cdot \left(\frac{\sigma_i}{f_{y,\theta}} \right)^{n_\theta} \quad (1)$$

σ_i , ε_i : represent the stress and corresponding strain, respectively, at temperature θ

n_θ : coefficient that enables the curvature to be adjusted;

$\beta = 6/7$.

Outinen et al. (1997) proposed simple formulas to determine the parameters of the Ramberg-Osgood equation (2,3,4) based on the test results for steel grade S355, as follows:

For $200^\circ\text{C} \leq \theta \leq 700^\circ\text{C}$:

$$E_\theta = 263000 - 325 \theta \quad (2)$$

For $200^\circ\text{C} \leq \theta \leq 700^\circ\text{C}$:

$$f_{y,\theta} = 352 - 0.54(\theta - 200) \quad (3)$$

For $200^\circ\text{C} \leq \theta \leq 700^\circ\text{C}$:

$$n_\theta = 0.000231 \theta^2 - 0.231 \theta + 62.5 \quad (4)$$

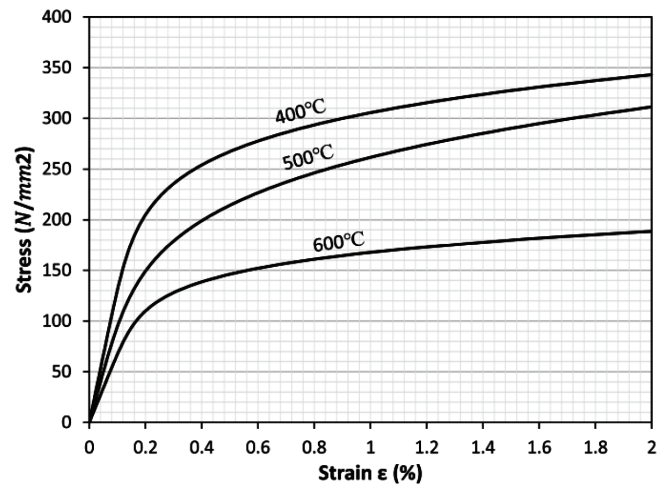


Fig. 3 Modified Ramberg-Osgood material model for S355 steel

3 NUMERICAL MODEL

3.1 General

An advanced geometrically and materially nonlinear analysis with imperfections included (GMNIA) using ABAQUS software (ABAQUS 3DEXPERIENCE r2018x) was used for calculating the ultimate load capacity of the members studied. The validation of this program was published in (Hajdú and Papp, 2018) and (Hajdú, 2021). The members are modelled using the general purpose S4R shell elements.

3.2 Study program

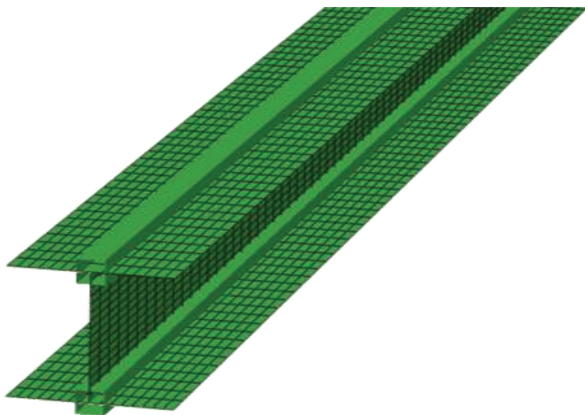
Four sections were studied to recognize the effect of material models on steel members with different sections subjected to an axial force or a bending moment. The dimensions of these cross-sections are presented in Table 2.

Tab. 2 Characteristics of different sections investigated in this paper

Cross-section	h	b	t_w	t_f	r
IPE160	160	82	5.0	7.4	9
IPE180	180	91	5.3	8.0	9
HE100B	100	100	6.0	10.0	12
HE500B	500	300	14.5	28.0	27

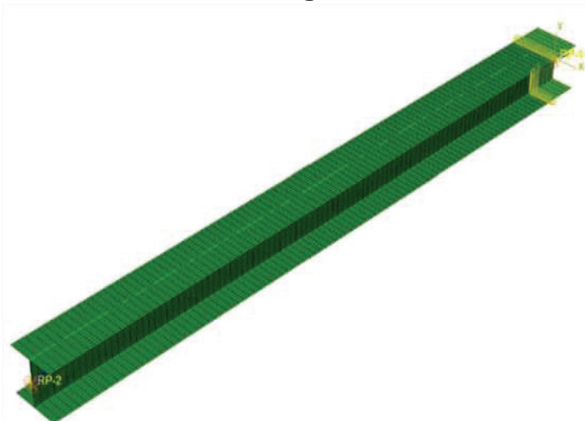
3.3 Meshing size

The mesh size was determined to be 16 elements in the flange, and 16 elements in the web depth, while along the member's length, the size of the elements was 20 mm, as shown in Fig. 4.

**Fig. 4** Meshing of the ABAQUS model

3.4 Load and boundary conditions

For the sake of simplicity, the members investigated in this study were simply-supported; i.e., two reference points, one at each end, were coupled with the nodes of both end surfaces of the members using kinematic coupling restraints. Then, these reference points were restrained against all degrees of freedom except for the displacement in the direction of the load applied at the loaded end and the rotations about the axes of the buckling at both ends, as shown in Fig. 5.

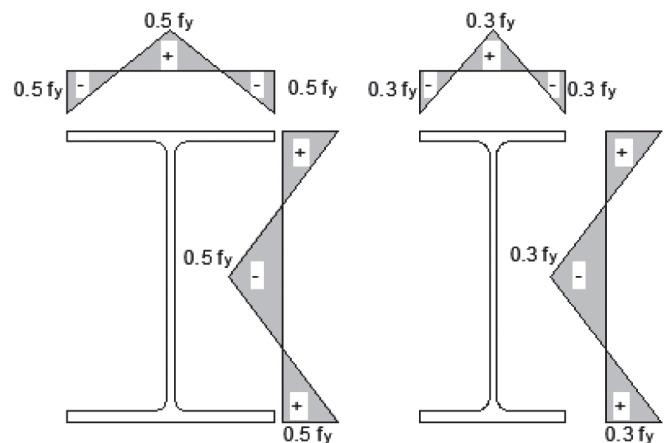
**Fig. 5** ABAQUS model: Boundary conditions and load application

The load was modelled by applying the distributed forces on the flanges and on the web of the loaded end using the modified RIKS tool, which is available in the ABAQUS library (ABAQUS, 2018).

3.5 Imperfections

The residual stresses were taken into account by introducing the ECCS type model for hot-rolled cross-sections (ECCS, 1976) into the numerical model. The amplitude of the initial stress depends on the height-to-width ratio of the section investigated, as shown in Fig. 6.

The initial geometric imperfections of the members were taken into account by performing a linear buckling analysis (LBA) on the perfect prismatic member with the given boundary conditions; then the relevant normalized global buckling mode was extracted. Thus, the first global buckling mode shape derived from the linear buckling analysis was introduced into the non-linear finite element model (GMNIA), and multiplied by the amplitude of the initial geometrical imperfection. The nodal coordinates of the model were updated by adding the nodal imperfections established. The amplitude of the initial geometrical imperfection of the column is considered to be equal to $L/1000$, where L is the member length. The amplitude of the initial geometrical imperfection of the column is taken to be $L/1000$, which is widely used in the literature, and corresponds to 75% of the recommended tolerance value of $L/750$ for a steel column in Annex D of EN1090-2:2008 (BS EN 1090 2, 2008).

**Fig. 6** Considered residual stress patterns for $h/b < 1.2$ (left) and $h/b > 1.2$ (right) [16]

It is worthwhile mentioning that in the finite element models, the true stress and plastic strain were adopted instead of the engineering stress and strain. Therefore, the following equations were used to represent the relationship between the true stress and plastic strain:

$$\sigma_{true} = \sigma_{nom}(1 + \varepsilon_{nom}) \quad (5)$$

$$\varepsilon_{true} = \ln(1 + \varepsilon_{nom}) \quad (6)$$

$$\varepsilon_{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (7)$$

Where:

- σ_{true} : true stress
- σ_{nom} : engineering or nominal stress
- ε_{nom} : engineering or nominal strain
- ε_{true} : true strain
- ε_{pl} : plastic strain

4 RESULTS

4.1 Comparison with the numerical simulation given by Prachar et al.

A number of class 4 (slender) welded steel beams were investigated numerically and experimentally at elevated temperatures by (Prachar et al., 2016). The two material models investigated in this paper were employed to simulate a heated Class 4 steel beam under lateral torsional buckling at 450 °C presented in (Pauli, 2012) as (TEST 6). It is worthwhile mentioning that in this model, the yield strength is based on the strain at 0.2% proof strength for both materials used. The dimensions of the cross-section of the beam along with the capacities calculated by Prachar et al., and by the numerical model described above incorporating the two material models are presented in **Table 3**.

Tab. 3 Numerical results using the two material models

NO.	Cross-section (mm)		Load capacities (kN)		
	$h_w \times t_w$	$b_f \times t_f$	Prachar et al.	GMNIA (EN1993-1-2)	GMNIA (R-O)
Test 6	446 X 4	150 X 7	151.84	151.63	149.14

The total load-displacement curves of the same numerical simulations are shown in **Fig. 7**. Based on the results presented in **Table 3** and **Fig. 7**, it can be said that adopting the modified Ramberg-Osgood material model in the numerical model results in a capacity similar to that calculated using the EN1993-1-2. However, there is a clear difference in the deflection, which is due to the difference in the Young's modulus value, as follows:

According to EN1993-1-2, the Young's modulus at 450 °C is $E_\theta = 136500 \text{ N/mm}^2$; while according to the test results given by (Outinen et al., 1997), $E_\theta = 116750 \text{ N/mm}^2$.

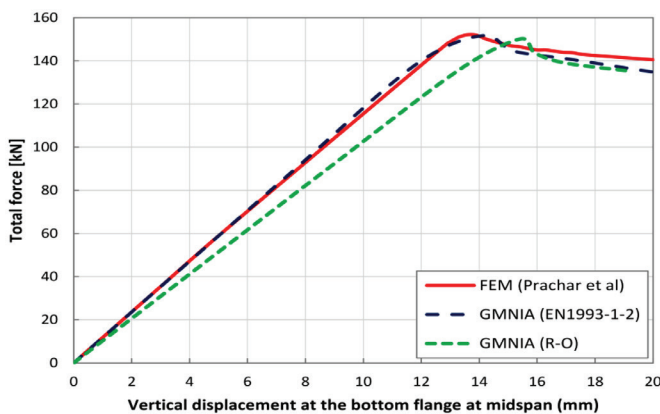


Fig. 7 Comparison of the buckling capacity-displacement curves of the beam investigated

4.2 Comparative study of columns

Following the comparison of the lateral torsional buckling of the steel beam presented in the previous section, the two material stress-strain curves were incorporated in the ABAQUS model to calculate the capacities (N) of steel compressed members in a fire with varying member lengths (slenderness ratios) and under three different temperatures.

Fig. 8 shows the results of the numerical models for the columns made of IPE160 and HE100B cross-sections at three different temperatures (400 °C, 500 °C, and 600 °C), where $N_{pl,t,Rd}$ is the plastic compressive resistance of the cross-section at elevated temperatures.

It can be seen that adopting the Ramberg-Osgood model can result in similar buckling capacities to those predicted by using the EN1993-1-2 material model for steel columns at 400°C and 500°C. However, for columns under 600°C, the Ramberg-Osgood model overestimates the buckling capacities for various slenderness ratios and cross-sections.

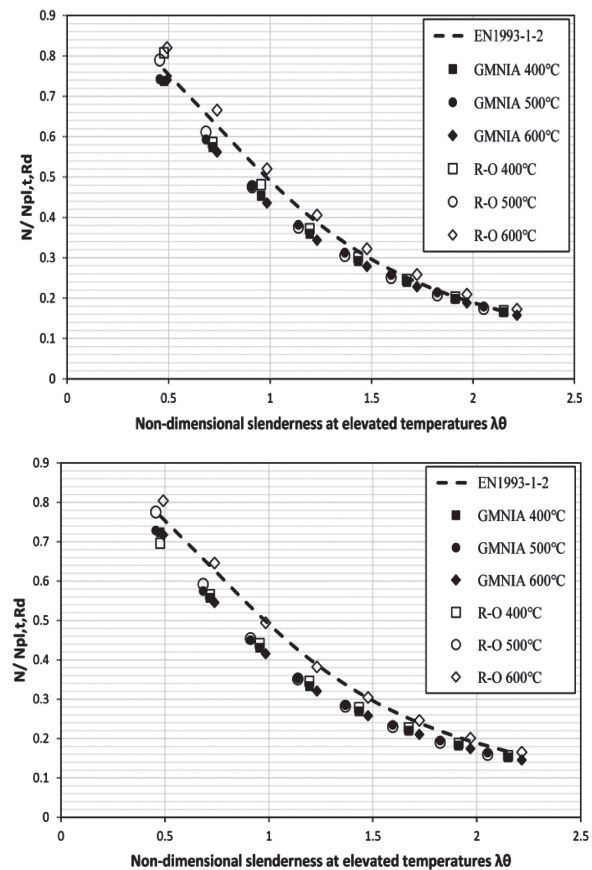


Fig. 8 Comparison of the buckling capacities for columns made of IPE160 (above) and HE100B (under)

4.3 Comparative study of beams

In this section, the two material models were used for calculating the buckling capacities of steel members under pure bending.

The results of the comparison of the lateral torsional buckling capacities (M) for members made of IPE180 and HE500B cross-sections at three different temperatures 400 °C, 500 °C, and 600 °C are presented in **Fig. 9**, where $M_{pl,t,Rd}$ is the plastic

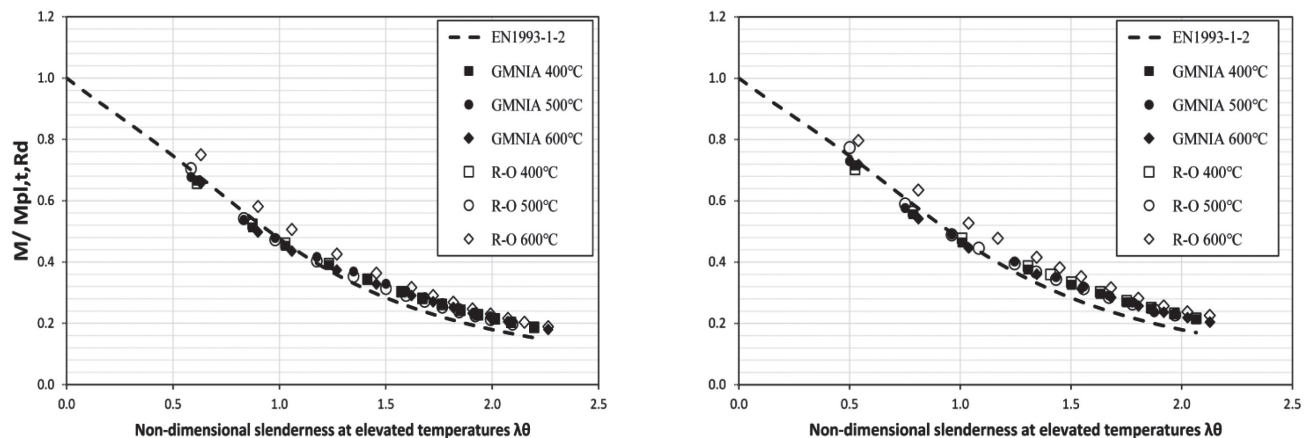


Fig. 9 Comparison of the buckling capacities for columns made of IPE160(Left) and HE100B (Right)

moment resistance of the cross-section at elevated temperatures.

Again, it can be seen that the two material models lead to almost the same buckling resistances for steel beams at 400°C and 500°C. However, for the beams at 600°C, the Ramberg-Osgood material model overestimates the buckling capacities compared to the capacities calculated with the EN1993-1-2 material model mainly for short beams, while this difference reduces for long beams (1.5), where both material models lead to similar capacities.

5 CONCLUSION

This paper presents a numerical investigation of the verification of the Ramberg-Osgood material model with parameters calculated based on a transient test achieved at the Helsinki University of Technology. A finite element model was developed and verified; then three different cases were investigated, i.e., a Class 4 steel beam at 450°C, a comparative study on columns with different slenderness ratios and three different temperatures (400, 500, 600°C), and a similar study on beams. It can be concluded that the modified Ramberg-Osgood leads to buckling capacities that correspond to those given by adopting the EN1993-1-2 material model for beams and columns at 400°C and 500°C. However, at 600°C, the Ramberg-Osgood model overestimates the buckling capacities for most cases. Therefore, the parameters should be modified, and more conservative values should be adopted.

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Disclosure statement

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

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COMPARATIVE STUDY OF THE LIGHTENING OF CONCRETE THROUGH THE USE OF RECYCLED TIRE RUBBER AND EXPANDED CLAY AGGREGATE

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Abstract

The waste of tire rubber is characterized by the generation of huge amounts, which is annually discarded. This waste causes problems for human health and the environment. Thus, the valorization of recycled tire rubber in concrete production is considered as an alternative and sustainable solution. This experimental work is a comparative study, which aims to investigate the possibility of using recycled tire rubber as a fine aggregate in the production of lightweight concrete. To achieve this objective, two types of lightweight concrete containing (35, 70 and 100%) of expanded clay and (5, 10 and 15%) of recycled tire rubber as a fine aggregate were produced. Their fresh properties were investigated using slump, density and air content tests, while the hardened properties were investigated by compressive strength, flexural strength, ultrasonic pulse velocity, and water absorption tests. The results obtained showed that the utilization of both expanded clay aggregate and recycled tire rubber leads to a reduction in density and mechanical strength but increases the porosity of lightweight concrete. However, the lightweight concrete containing the expanded clay aggregate had a higher efficiency factor than those containing recycled tire rubber.

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Key words

- Lightweight concrete,
- Expanded clay aggregate,
- Recycled tire rubber,
- Properties,
- Valorization.

1 INTRODUCTION

The production of lightweight concrete (LWC) is an important technological field; LWC is characterized by a very low density ($< 2000 \text{ kg/m}^3$) (Andressa et al., 2020). The lower density of LWC leads to the reduction of foundation sizes and structural elements (Katrin Schumacher et al., 2020; Basha et al., 2020). Due to its lower density, LWC is also defined by enhanced thermal and sound insulation (Sousa et al., 2004; Hubertova and Hela, 2009; Grabois et al., 2016), higher specific strength and increased fire resistance (Amin et al., 2019; Xi et al., 2019). The performance of LWC is characterized by an important parameter called the “efficiency factor” (EF), which represents the ratio between the density and compressive strength of LWC (Andressa et al., 2020). The produc-

tion of LWC requires the use of natural or artificial lightweight aggregates. The aggregates mostly used for the production of LWC are expanded clay, sintered fly ash, and expanded shale (Xi et al., 2019). Expanded clay aggregates (ECA) are artificial lightweight aggregates, which are produced by expanding natural clay at a high temperature (around 1200°C) in a rotary kiln (Ardakani and Yazdanie, 2014; Ayati et al., 2018; Rashed 2018). The utilization of ECA, which has better compressive strength than other lightweight aggregates, decreases the cost of reinforced steel by 20% and reduces the cost of heating and cooling by 50% (Pavithra and Jerosia, 2018; Vijayalakshmi and Ramanagopal, 2018; Sellakkannu and Tamilarasan, 2016). However, LWC containing ECA exhibits a decrease in compressive strength (Sellakkannu and Tamilarasan, 2016; Corinaldesi and Moriconi, 2015). (Bogas and Nogueira,

2014) added that the use of ECA as a coarse aggregate decreases the tensile strength of LWC by 15-20%. (Sivakumar and Kameshwari, 2015) mentioned that an increase in ECA content progressively decreases the compressive and flexural strength of LWC. (Lakshmi Sravya et al., 2020) recommended the use of cementitious mineral admixtures to avoid a significant drop in the mechanical properties of LWC. (Serkan, 2009) showed that the adding of 10% of fly ash increases the compressive strength and tensile strength of LWC by 8 and 9% respectively. The studies carried out by (Józwiak-Niedzwiedzka, 2005; Shankar, 2016; Shebannavar et al., 2015) showed that ECA had a beneficial effect on the workability of concrete and that concrete containing ECA was more workable than normal weight concrete. (Bogas et al., 2015; Salem et al., 2014) reported that the adding of ECA increases the absorption of LWC by about 55%. (Bogas and Gomes, 2015; Zhang and Gjorv, 1991; Hobbs, 1999) found that ECA harms the durability and resistance to chloride of LWC.

On the other hand, huge amounts of used tire rubber (TR) are generated annually. (Tomas and Gupta, 2016) reported that about 1 billion tons of used TR is produced in the world annually and that more than half of this waste is discarded in landfills. (Azevedo et al., 2012) added that by the year 2030, the amount of TR waste will reach 6 billion tons. This waste causes serious problems for public health and the environment (storage problems, fire risks, and the dissemination of insects). Several studies have been carried out to evaluate the possibility of reusing recycled tire rubber (RTR) in concrete production to eliminate TR waste from the environment. (Bravo and Brito, 2012; Mhaya et al., 2020) concluded that RTR decreases the workability of concrete. (Su et al., 2015) added that larger particles of RTR reduced the workability of concrete less than finer particles. (Siddika et al., 2019) reported that the density of concrete decreased with an increase in RTR content. (Aslani et al., 2018; Si et al., 2018; Zhang et al., 2015; Najim and hall, 2012) noted that the adding of RTR reduced the mechanical strength of concrete. The compressive strength dropped by 90% with the total substitution of fine aggregates by RTR (Batayneh et al., 2008). (Bisht and Ramana, 2017) added that the presence of RTR in concrete composition also reduced the elastic modulus of concrete. (Thomas et al., 2014) showed that up to 7.5% RTR can be used as a fine aggregate without a significant decrease in strength. To avoid a significant drop in the strength of concrete containing RTR, several studies recommended the use of RTR with other cementitious additions such as silica fume (AbdelAleem and Hassan, 2018; Gesoglu and Guneyisi, 2007), fly ash (Erhan, 2010), and blast furnace slag (Mhaya et al., 2020). (Yung et al., 2013) noted that 5% of RTR powder provided optimal durability properties for concrete. (Thomas et al., 2015) found that RTR improved the resistance of concrete to acid attacks. (Thomas et al., 2016b) showed that RTR increased the water absorption of concrete. However, it has been shown that the reuse of RTR improves the ductility, abrasion resistance (Li et al., 2018), thermal insulation, and flexibility of concrete (Lv et al., 2015).

2 SIGNIFICANCE OF RESEARCH

Concrete is an important construction material and is among the most widely used materials in the world (Andressa et al., 2020). It has been estimated that 12 billion tons of con-

crete are produced annually. It has also been estimated that the need for concrete will increase to 18 billion tons by the year 2050 (Singh et al 2017). The higher demand for concrete has led to intensive exploitation of the raw materials used in their production. The utilization of waste and by-products in concrete production is considered as an effective and sustainable solution, which could protect the environment and public health, reduction the consumption of raw materials, and save energy. This work aims to study the possibility of reusing RTR as lightweight aggregates in LWC production through a comparative study examining the performance of LWC containing RTR and other LWC containing ECA. Knowing that ECA production needs a higher amount of energy, the substitution of ECA by RTR could lead to the production of eco-friendly LWC.

3 MATERIALS USED

3.1 Cement

Portland cement (PC) CEMI 42.5 was used in this study. The specific density and specific surface area of PC were 3.11 g/cm³ and 4308 cm²/g respectively. The chemical and mineralogical compositions of PC used are illustrated in **Tables 1 and 2**.

Tab. 1 Chemical composition of PC

Chemical properties (%)	PC
CaO	64.29
Al ₂ O ₃	4.74
Fe ₂ O ₃	2.89
SiO ₂	18.54
MgO	1.34
Na ₂ O	0.24
K ₂ O	0.57
Cl-	0.05
SO ₃	3.01
Loss in ignition	3.31

Tab. 2 Mineralogical composition of PC

Mineralogical Composition (%)	PC
C ₄ AF	11.25
C ₃ A	7.50
C ₃ S	60.45
C ₂ S	12.24

3.2 Aggregates

Coarse crushed limestone sand (LS) with a specific gravity of 2.64, a water absorption of 0.94%, and a finesse modulus of 3.25 was used as a fine aggregate. Fine siliceous dune

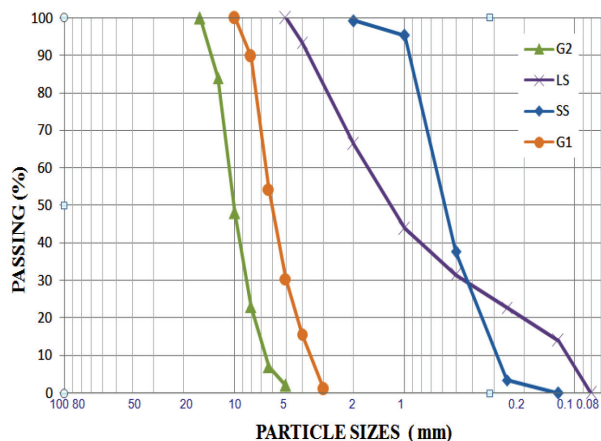


Fig. 1 Distribution of particle sizes of aggregates used

sand (SS) with a specific gravity of 2.65, a water absorption of 0.48%, and a finesse modulus of 1.64 was used to correct the grain size distribution of LS. Two crushed limestone gravels (G1 and G2) with nominal maximum sizes of 8 and 15 mm respectively, a specific density of 2.69 g/cm³, and a water absorption of 0.94% were used as coarse aggregates (CA). Expanded clay aggregate (ECA) with a nominal maximum size and specific density of 4 mm and 1.68 g/cm³ respectively was used as a lightweight aggregate. The particle distribution of the aggregates is presented in Fig. 1.

3.3 Recycled tire rubber (RTR)

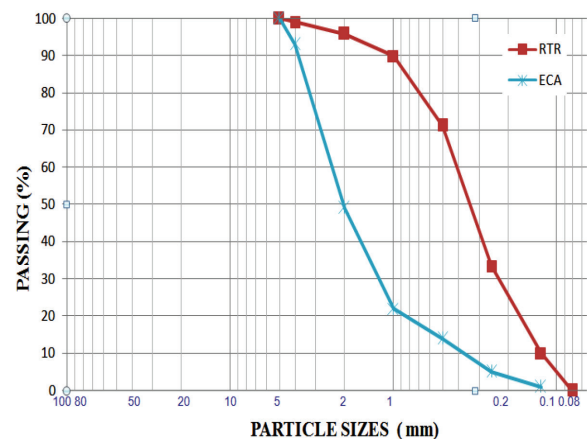


Fig. 2 RTR used

The RTR used in this study was obtained by cutting end-of-life tires in small particles, followed by washing them (Fig. 2). The RTR had a maximum size of 4 mm, a specific density of 1.10 g/cm³, and a water absorption of 1.65%. The particle distribution of RTR is presented in Fig. 1.

3.4 Admixture

To obtain the desired workability, visocrete 665 based on polycarboxylate with a brown color, a density of 1.08 g/cm³, and a PH of 5 was used as a superplasticizer (SP).



4 MIXING PROCEDURE AND TEST METHODS

Two types of concrete were produced to carry out this study. The first type contains ECA as a lightweight aggregate, while the second type contains RTR. The concrete mix proportions are provided in Table 3. The contents of PC, water, and SP were kept constant. ECA and RTR were introduced in the concrete composition by the volumetric substitution of the normal fine aggregate. The substitution rates chosen were 35, 70, and 100% for the ECA and 5, 10, and 15% for the RTR aggregates. The maximum RTR rate was limited to 15% to avoid a significant drop in the mechanical strength of the mixtures.

The solid constituents were mixed for three minutes. Then, the water and SP were mixed with the other constituents for three more minutes. After mixing, the mixture was introduced and vibrated into standard steel molds according to NF P18-405. Cube samples (150×150×150 mm) were used for the compressive strength, pulse velocity and water absorption tests. Prismatic samples (70×70×280 mm) were used for the flexural strength. After 24 hours of molding, the samples were demolded and conserved in water curing (at 23 °C and 100% relative humidity) until their testing days.



Fig. 3 Preparation of specimens for test.

The fresh and hardened property tests were performed according to the standards mentioned below. For each test, the results obtained represent the average of three readings.

Tab. 3 LWC mixtures

	G1 (Kg/m ³)	G2 (Kg/m ³)	SS (Kg/m ³)	LS (Kg/m ³)	PC (Kg/m ³)	SP (L/m ³)	Water (L/m ³)	ECA (Kg/m ³)	RTR (Kg/m ³)
C0	260	748	150	701	350	2,1	147	0	0
ECAC35	260	748	150	436	350	2,1	147	134	0
ECAC70	260	748	150	214	350	2,1	147	268	0
ECAC100	260	748	150	00	350	2,1	147	382	0
RTRC5	260	748	150	674	350	2,1	147	00	16
RTRC10	260	748	150	637	350	2,1	147	00	32
RTRC15	260	748	150	599	350	2,1	147	00	47

- ✓ Slump test NF EN 12350-2
- ✓ Bulk density in fresh state NF EN 12350-6
- ✓ Air content NF EN12350-7
- ✓ Compressive strength NF EN 12390-3
- ✓ Flexural strength NF EN 12390-5
- ✓ Pulse velocity NF EN 12504-4
- ✓ Water absorption NBN B15-215

5 DISCUSSION OF RESULTS

5.1 Workability

The workability results are shown in Fig. 4 for the concrete mixtures containing the different volume fractions of ECA and RTR. As can be seen, the adding of ECA insignificantly affects the workability of LWC. The workability decreases from 21 to 20 cm when the ECA content increases from 0 to 100%. The non-reduction of the workability of LWC containing ECA was attributed to the rounded surface of ECA, which facilitates the LWC flow (Rashed et al., 2018). However, the LWC mixtures containing RTR showed a significant decrease when the RTR rate exceeds 10%. For the 15% of the RTR content, the workability dropped by 14.28%. This reduction was explained by the rough nature of the RTR grains, which increase the friction between the LWC components (Holmes et al., 2014; Su et al., 2015).

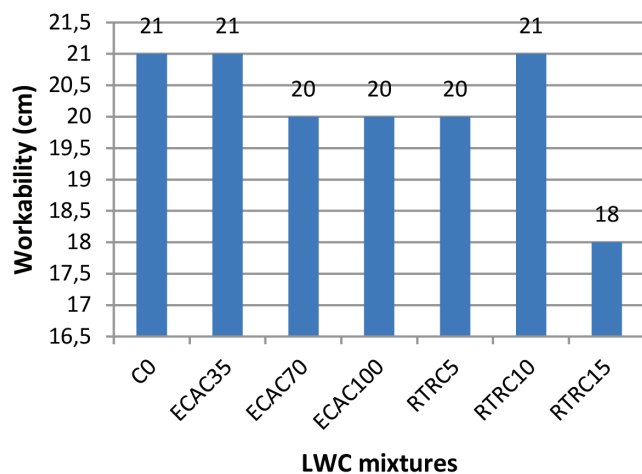


Fig. 4 Workability of LWC mixtures

5.2 Bulk density

Fig. 5 shows the bulk density values of the LWC mixtures. The increase in ECA and RTR contents gradually decreases the density of LWC. This decrease is more pronounced for LWC containing ECA. The bulk density decreases by 13.26% and 6.79% when 100% and 15% of ECA and RTR are used respectively. The bulk density of concrete mainly depends on the density and the rates of its components (Siddique et al., 2004). The utilization of ECA and RTR, which have lower densities than the other components, decreases the bulk density of the concrete. This decrease was more pronounced with LWC containing ECA due to the higher contents of ECA in the LWC composition compared to that of LWC containing RTR.

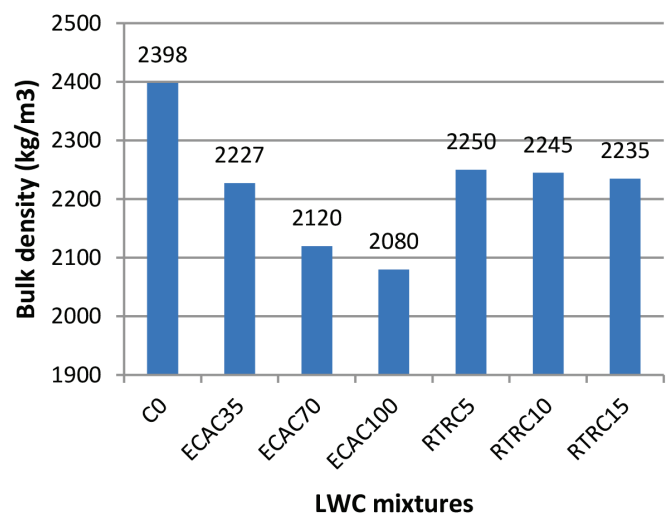


Fig. 5 Bulk density of LWC mixtures

5.3 Air content

The air content values are shown in Fig. 6. As can be seen, increasing the ECA and RTR content in the LWC composition progressively increases the air content values. It can also be seen that a maximum increase of 171% was recorded with LWC containing 100% of ECA, while LWC containing 15% of RTR exhibited an increase by 66.66%. The increase in the air content value of concrete containing RTR was also shown by (Ling et al., 2010; Siddique et al., 2004) and attributed to the fact that RTR has a non-polar nature, which traps the air bubbles in the composition of the concrete. The increase in the

air content of LC containing ECA is due to the porous nature of ECA, which facilitates the air entrapment (Rashad 2018)

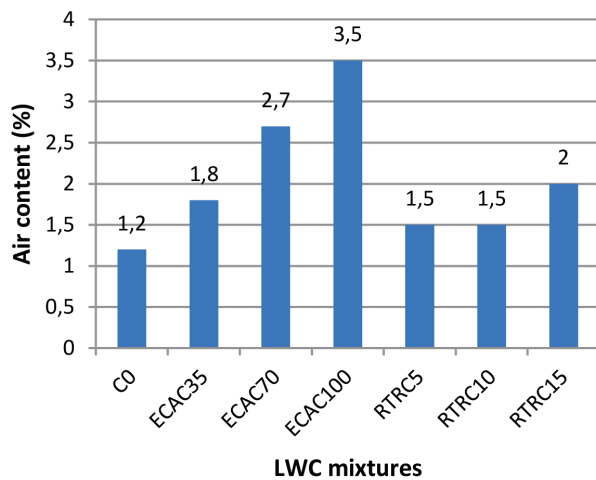


Fig. 6 Air content of LWC mixtures

5.4 Compressive strength

The variations of the compressive strength of LWC containing different volumes of ECA and RTR at 7, 28, and 56 days is presented in Fig. 7.

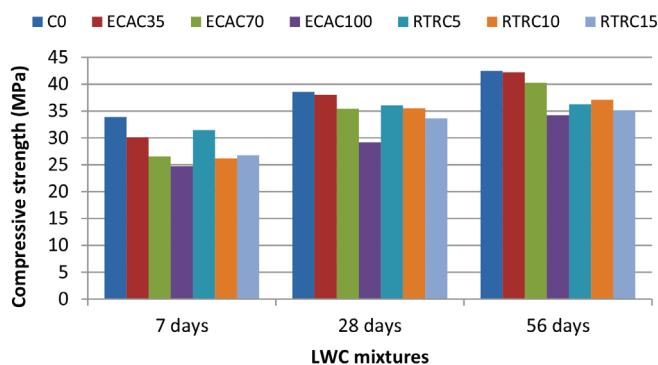


Fig. 7 Compressive strength of LWC mixtures

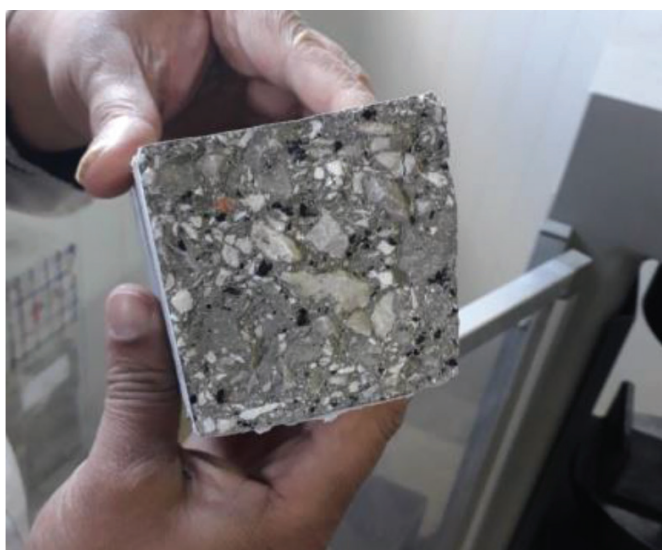


Fig. 8 Failure mode of LWC containing RTR.

All the concrete mixtures show a progressive reduction in compressive strength with the increase in the ECA and RTR contents. The reduction in the compressive strength reaches 23.15 and 12.83% for the LWC containing 100 and 15% of ECA and RTR respectively at 28 days. (Bogas and Gomes, 2013) explained the drop in the compressive strength of LWC containing ECA by the fact that 1) ECA has a lower crushing strength, which negatively affects the compressive strength of LWC, 2) ECA has a rounded shape, which decrease the bond with the surrounding paste. (Gangian et al., 2009) attributed the reduction of the compressive strength of concrete containing RTR by the lack of adhesion between the cement paste and RTR (Fig. 8).

5.5 Efficiency factor

The efficiency factor (EF) is a parameter calculated using the ultimate strength per density. The variations of EF of LWC mixtures are illustrated in Fig. 9. It can be seen that the addition of 35 and 70% of ECA in an LWC composition leads to an increase in EF by 6.88 and 7.16% respectively. For a further increase in the ECA content, the EF tends to decrease. However, all the LWC mixtures containing RTR exhibit a gradual decrease in EF. The EF decreases from 17.72 to 15.71 when the RTR content increases from 0 to 15%. This decrease is explained by the significant reduction of the compressive strength of LWC containing RTR despite its low content.

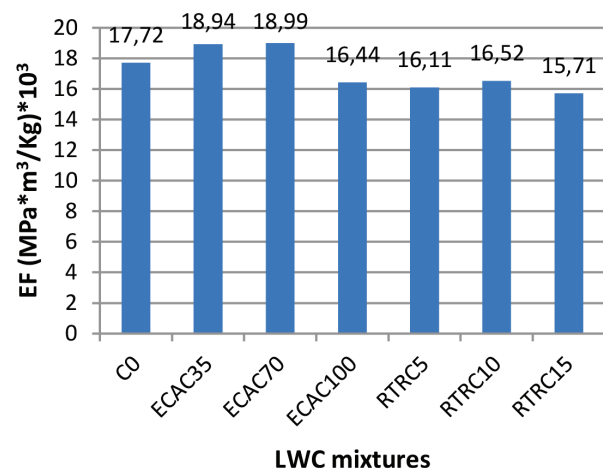


Fig. 9 Efficiency factor of LWC mixtures

5.6 Flexural strength

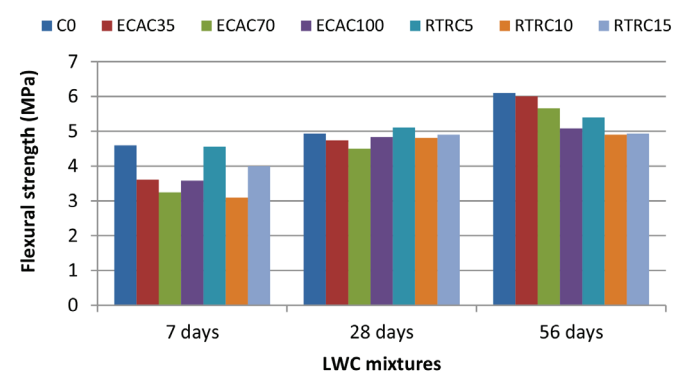


Fig. 10 Flexural strength of LWC mixtures

The results of the flexural strength tests are illustrated in **Fig. 10**. Like the compressive strength, the flexural strength of LWC decreases with the addition of ECA and RTR in the LWC composition. At 56 days, the flexural strength decreases from 6.01 to 5.08 and 4.93 MPa, when 100 and 15% of ECA and RTR were used respectively, i.e., a decrease of 15.47 and 17.97% respectively. This decrease is due to the rounded nature of ECA and the lack of adhesion of RTR with cement paste (Bogas and Gomes, 2013; Gangian et al., 2009).

5.7 Ultrasonic pulse velocity (UPV)

Fig. 11 shows the ultrasonic pulse velocity (UPV) of LWC mixtures at 28 days. The control mixture had an ultrasonic pulse velocity of 4.87 km/s, whereas all the LWC containing ECA and RTR had a lower UPV compared with the control concrete. It should be noted that according to ASTM C597, which considers the concrete to be of very good quality when the ultrasonic pulse velocity is higher than 4.5 Km/s, all the LWC mixtures are classified as having a very good quality.

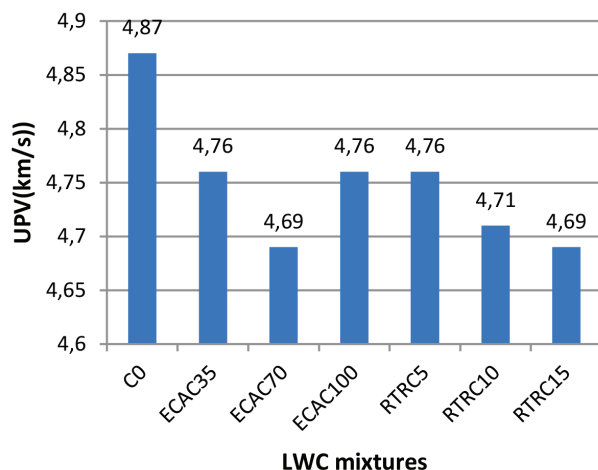


Fig. 11 UPV of LWC mixtures

5.8 Water absorption

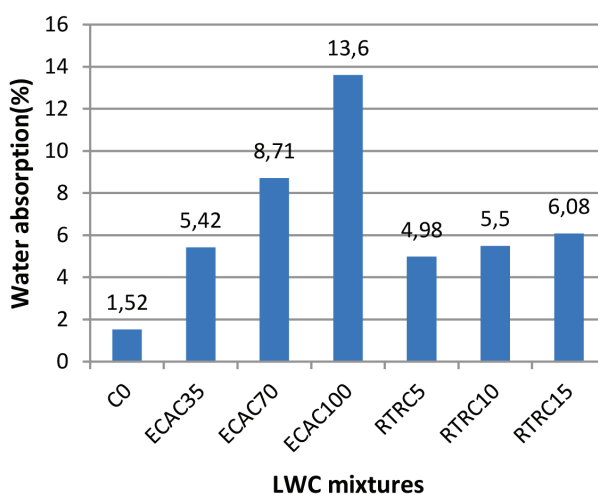


Fig. 12 Water absorption of LWC mixtures

The water absorption of LWC mixtures constitutes a major factor that affects their durability and represents the volume of the permeable pores of the LWC mixtures and their connectivity inside the concrete. The water absorption values of FRC mixtures are presented in **Fig. 12**. As can be seen, all the LWC mixtures have water absorption values higher than that of the control concrete. The water absorption gradually increases with the increase in the ECA and RTR contents. The water absorption of the control concrete was 1.52%, while the maximum water absorption values were 13.60 and 6.08% for the LWC containing ECA and RTR, respectively. It can also be seen that the LWC containing 100% of ECA exhibits a maximum increase of the water absorption value by 13.60% compared to that of the control concrete. (Rashed, 2018) attributed the increase in the water absorption of LWC containing ECA to the higher water absorption and the nature of the open voids of ECA, compared with ordinary aggregates, while the increase of water absorption of LWC containing RTR was explained by the lack of bonding between the RTR and cement paste, which facilitates the penetration of water in the LWC composition (Gangian et al., 2009). According to (CEB-FIP 1989), concretes having water absorption of less than 3%, between 3 and 5%, and higher than 5% can be classified as good, average and poor quality. According to this classification, the LWC containing 5% of RTR exhibits an average concrete quality, whereas all the other LWC mixtures have a poor quality.

5.9 Dynamic elastic modulus

The dynamic elastic modulus of the concrete mixtures was evaluated using the following model (Uysal and Yilmaz 2011):

$$E = 10^5 \times V^2 \times \left(\frac{\Delta}{g}\right) \quad (1)$$

E is the dynamic elastic modulus (GPa); V is the ultrasonic pulse velocity (m/s); Δ is the bulk density of concrete (kg/m^3); and g is the gravity acceleration (m/s^2).

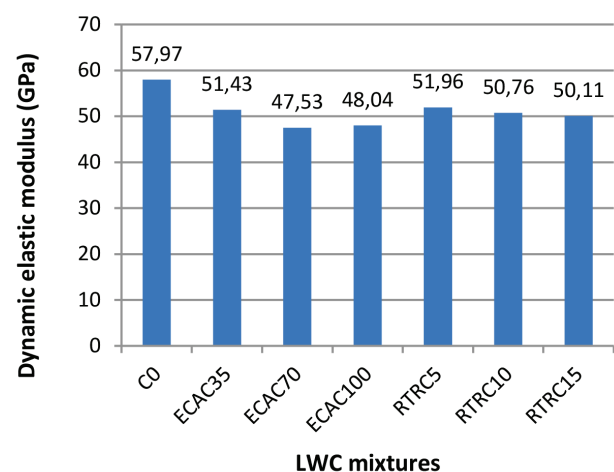


Fig. 13 Dynamic elastic modulus of LWC mixtures

The dynamic elastic modulus values of the LWC mixtures are presented in **Fig. 13**. As with the compressive strength, the dynamic elastic modulus decreases with the increase in ECA and RTR in the LWC mixtures. The lower value was obtained

with 10% of ECA, which exhibits a decrease of 18% in the dynamic elastic modulus, while the LWC containing 15% RTR exhibits the lower dynamic elastic modulus of 13.55% for the LWC containing RTR. This reduction in the dynamic elastic modulus was mainly due to the decrease in the compressive strength values (Li et al., 2018).

5 CONCLUSIONS

This work is aimed at studying the possibility of reusing RTR as a fine aggregate to produce LWC through a comparative study between the properties of LWC mixtures containing ECA and RTR as lightweight aggregates. Based on the experimental results, the following conclusions can be drawn:

- The addition of ECA insignificantly affects the workability of LWC, while the RTR decreases the workability when its content exceeds 10%.
- The air content of LWC increases with the increasing ECA and RTR content. The higher value was recorded with 100% of ECA.
- Despite its limited contents compared to the ECA contents, the addition of RTR in the LWC composition leads to a significant reduction in the compressive strength. This significant reduction leads to a decrease in the efficiency factor values of LWC containing RTR compared to that of LWC containing ECA.
- Both ECA and RTR increase the water absorption of LWC. The increase in the water absorption value was more pronounced with LWC containing ECA.

Finally, it can be noted that the use of recycled tire rubber as a lightweight aggregate in the production of lightweight concrete presents promising ecological and technical interests. From the results obtained, the LWC containing RTR can be applied in the fabrication of non-load bearing elements when compressive strength is not required (sealing element, dividing walls, and cleanliness of the concrete), especially when the RTR content exceeds 15%. It should also be noted that the utilization of RTR in the LWC composition leads to the production of eco-friendly materials.

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EVALUATION OF A MEASUREMENT TURBULENCE MODEL OF THE WIND PRESSURE ON THE RUIN OF A FORTIFIED TOWER

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Abstract

An analysis of the external pressure coefficient on the surface of a ruin in different flow directions is presented. The ruin has almost cube-like proportions with an open roof plane and a destroyed corner. Flow simulations were performed using 3D Time Steady RANS and compared with experimental results from the boundary layer wind tunnel at the Slovak University of Technology in Bratislava. The optimal turbulence model and internal mesh settings were selected based on their statistical evaluation. For an evaluation of the critical directions of the wind flow around the ruin, the values of the external wind pressure coefficient were obtained from the selected calculation model and settings.

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Key words

- Ruin protection,
- Wind tunnel,
- Computational fluid dynamics.

1 INTRODUCTION

In the context of monument protection, wind flow is of primary interest because of its destructive potential. As a destructive mechanism, it stresses the monument (a) mechanically and (b) chemically. Monuments are mechanically stressed by (aa) changing their surface integrity – for example, by grinding the surfaces of structures and removing elements of building materials loosened by freezing (e.g., plaster, mortar, or sedimentary rock layers in stone masonry); and by (ab) creating critical loads on structures, such as during extreme gales or storms, where not only elements prone to tearing off, such as roof coverings, but also stand-alone structures with compromised stability, are at risk. The wind contributes to the chemical stress on a monument (ba) by altering the water and gas chemical reactions caused by changes in pressure on it, or (bb) by transporting water, salts, sediments, and gaseous impurities (Sesana et al., 2021; Sabbioni et al., 2012; How, 2007).

The movement of the wind around a structure is greatly influenced by (a) the characteristics of the surrounding envi-

ronment and (b) the characteristics of the structure itself. The characteristics of an environment can be described (aa) on a landscape scale by its orography, elevation, and orientation in the direction of any flow (Oláh et al., 2013; Global Wind Atlas); (ab) on an urban scale by the type of environment causing frictional forces on its surface, as expressed by the extent of its roughness, the type and density of the urban environment in which the structure is located, and by other interfering objects, such as neighboring greenery; and (ac) by the orientation of the structure against the flow (Liu et al., 2018; STN EN 1991-1-4, 2006). On an architectural scale, the flow is influenced by the characteristics of the structure (ba) its proportion, which is given by the ratio of the height of the structure to its width or depth (Mou et al., 2017), (bb) its shape, which is given by the geometry of the ground plan base of the structure, the type of roofing and its height from the base (Singh and Roy, 2019; Fouad et al., 2018), (bc) the spectrum of the architectural and constructional details of any peripheral structures, and (bd) the spectrum of building materials of any peripheral structures, which could affect the flow in the immediate vicinity of the surfaces of the structure.

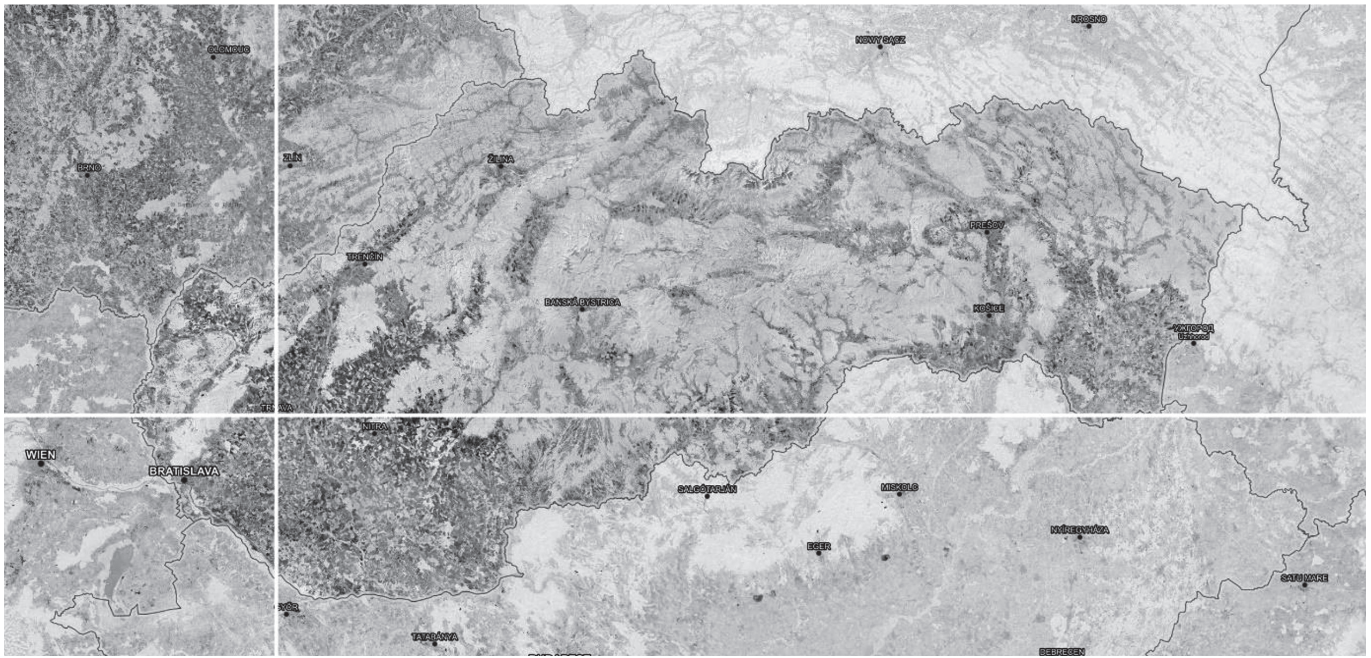


Fig. 1 Map of Slovakia; the cross is marking the location of the town of Trnava (©mapy.cz)

With Computational Fluid Dynamics (CFD), the behaviour of the wind can be predicted from either the perspective of its own destructive mechanisms or from the perspective of another mechanism distributed by it. The use of CFD is used (a) to simulate wind movement in a given environment, (b) to analyse historical wind flow in an environment to confirm hypotheses related to interpretation, and (c) to make conservation design decisions (Grau-Bové et al., 2019). The simulation of wind movement in a certain environment was applied in the search for the most exposed and weathered places of the Giza plateau and the Great Sphinx by calculating the distribution of the friction and pressure coefficients and supplementing them with wind speed calculations (Hussein and El-Shishiny, 2009). As an aid to decision-making, wind flow simulations were used to estimate the wind-driven sand erosion of the Cardo stone colonnade of the Hispano-Romanesque site of Baelo Caludia in Cadíz, Spain (Pineda and Iranzo, 2017).

(Grau-Bové et al., 2019) examined the unusual employment of CFD in the cultural heritage context for the purposes of developing this computational methodology. That use is more focused on case studies and assistance in solving practical problems. The study also mentions how rare it is for simulations to be validated by another research method, which would serve to verify and calibrate the computational model and possibly bring the computational methodology closer to real-world results. This is primarily due to the long-term nature of simulations or because the problem to be solved is limited to simulating very slow interior flows, which are difficult to replicate experimentally. More than the wind itself, other destructive mechanisms such as the temperature, humidity, dust particles, or gaseous impurities distributed by the wind are of greater interest.

Philosophically, the present paper considers monuments as physical foundations of culturally and ecologically sustainable development and as the preservation of the genius loci, as reflected on different spatial scales and origins (Vecco, 2020; Pilař, 2016; Antrop, 2006; D'Acci, 2019; Nádaská and Pilař,

2017; Ruhig, 2017; Bránický and Gregorová, 2022). In so doing, the paper uses an idealised model of the ruin of the south-eastern corner tower of the Trnava fortifications, which has previously been identified as a significant location of cultural value and is currently undergoing actualisation of the monumental presentation (Gregorová, 2022). The tower has also been identified as one of the locations potentially vulnerable to climate change (Poliak, 2021). The external pressure coefficient c_{pe} on the surface of the Trnava monument illustrates the flow and its impact on its substance. The calculations took place in the ANSYS Fluent software environment using alternatives that consider different flow directions, apply several types of turbulence models of the Reynolds-Averaged Navier-Stokes (RANS) turbulence, and change the internal settings of the domain and mesh of the calculations. A comparison is made between the calculations made at specific positions and the measurements taken in the Boundary Layer Wind Tunnel (BLWT) at the Slovak University of Technology in Bratislava (STU BA), to ensure the correct methodology in further research on the effectiveness of a roof in covering a ruin through its material protection. An optimal calculation model and alternative internal settings were selected using three statistical methods. Using the optimal calculation model, the most unfavourable flow directions around the ruin model were determined.

2 MATERIALS AND METHODS

2.1 Idealisation of the model of the selected ruin

The southeast corner tower of the fortification of the town of Trnava (Fig. 1), which is located between the grounds of the West Slovak Museum and the town amphitheatre (Fig. 2), is the subject of our research. Before losing its defensive functions and before its gradual destruction, it was a five-story building with an almost square base measuring approximate-



Fig. 2 Orthophoto of the town Trnava; the cross is marking the location of the southeast corner tower of the fortification (©mapy.cz)

ly 8×8 m and with walls approximately 2.6 m thick on the lowest floor. The first four floors date back to the Romanesque period, while the fifth floor was added during the Baroque period by filling in battlements and adding a pyramidal roof. Each tower was connected by walls and had approximately three floors, including a parapet (Gregorová et al., 1996). At present, it is preserved at a height of two full floors with the impression of a third floor, an inner corner destroyed almost to ground level, and serrated front sides (typical of the presentation of a monumental fortification). Only the eastern curtain up to the height of the upper corridor has been preserved. The tower and the curtain are both the same height, with their masonry crowns capped with a concrete plate with a slight overlap (Fig. 3).

A portion of the urban context and architectural detail needed to be abstracted from the actual situation for the purpose of this work - (a) the configuration of the surrounding terrain, which is replaced by a plane with an averaged level height; (b) the surrounding buildings and greenery, creating interferences in the run-up area of the flow; (c) sections of walls (curtains) connected directly to the structure of the ruin; (d) architectural details such as overlaps of the concrete slab on the crowns of the masonry, serration of the wall faces, and the openings of slotted embrasures; and (e) the differences in the construction materials of the masonry and the slab, which is protecting the head of the masonry (Fig. 3).

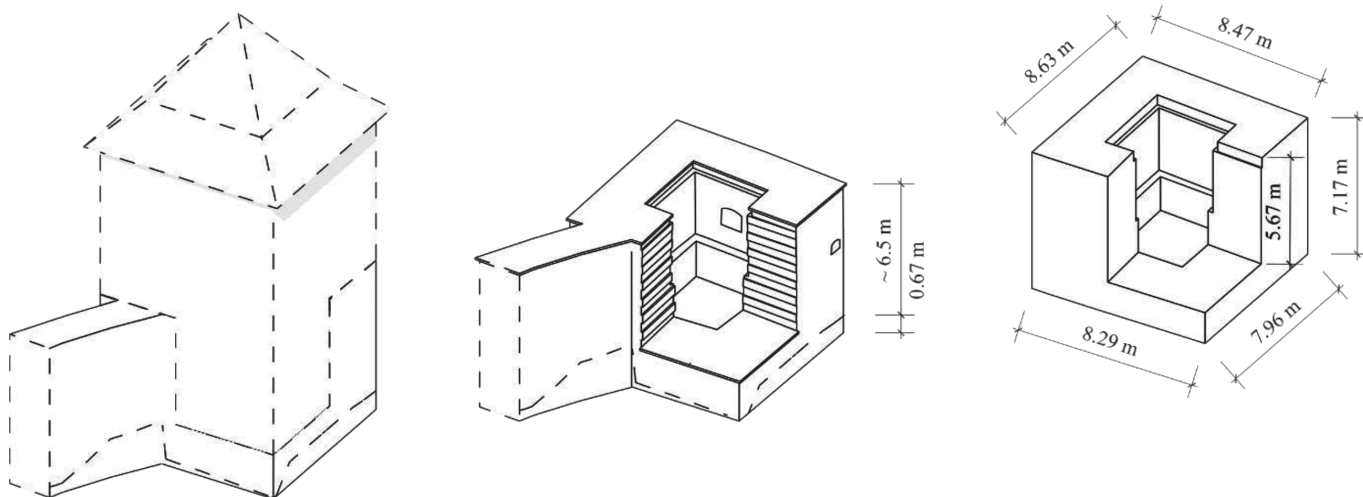


Fig. 3 Trnava's fortified tower of interest in its original, current, and idealised state

2.2 Experimental measurement in BLWT STU BA

The Boundary Layer Wind Tunnel (BLWT), a low-speed tunnel with a length of 26.2 m and clear dimensions of 2.6×1.6 m (Hubová and Lobotka, 2014), was used for the experimental measurement of the external pressure coefficient c_{pe} on the surface of the idealised ruin model. Based on the mean speed of the wind flow and the intensity of the turbulence, the roughness length $z_0 = 0.7$ m corresponds to the flow in a real-scale urbanized environment and the terrain categories III and IV according to the Eurocode (STN EN 1991-1-4, 2006). The external pressure coefficient c_{pe} was calculated according to relation (1) based on the difference between the local static pressure in tap p and the free stream mean static pressure p_s , air density $\rho = 1.164$ kg·m⁻³, and the reference flow velocity wind $v_{ref} = 7.871$ m·s⁻¹:

$$c_{pe} = \frac{p - p_s}{\frac{1}{2} \rho v_{ref}^2} \quad (1)$$

A 3D printer was used to print the physical model on a 1:390 scale in PLA with a 0.3 mm profile width, 0.3 mm layer height, and final wall thickness of 4 mm with 100% infill. The model

was fitted with 40 pressure taps at three heights: at the crown of the ruin's masonry as the highest horizontal plane, at two-thirds of the model's height, and at its foot in the open corner (Fig. 4). The flow was simulated in 16 directions in 22.5° steps.

2.3 Numerical calculation in CFD

The calculations took place in the ANSYS Fluent software environment in alternatives. The flow situations were simulated as pressure-based, time steady, and without a production limiter and curvature correction. The maximum residual value of all the quantities was set at 10^{-5} . The simulations were performed in double precision on a desktop computer with an Intel® Core™ i7-10700K @ 3.80 GHz processor and 32.0 GB DDR4 RAM.

The dimensions of the computational domain were based on the height H of the idealised model of the ruin on a scale of 1:390, i.e., on a scale identical to the dimensions of the physical model used for experimental measurements in the BLWT (Fig. 5) (Franke et al., 2007). The resulting volume of the computational domain was 19,965 m³.

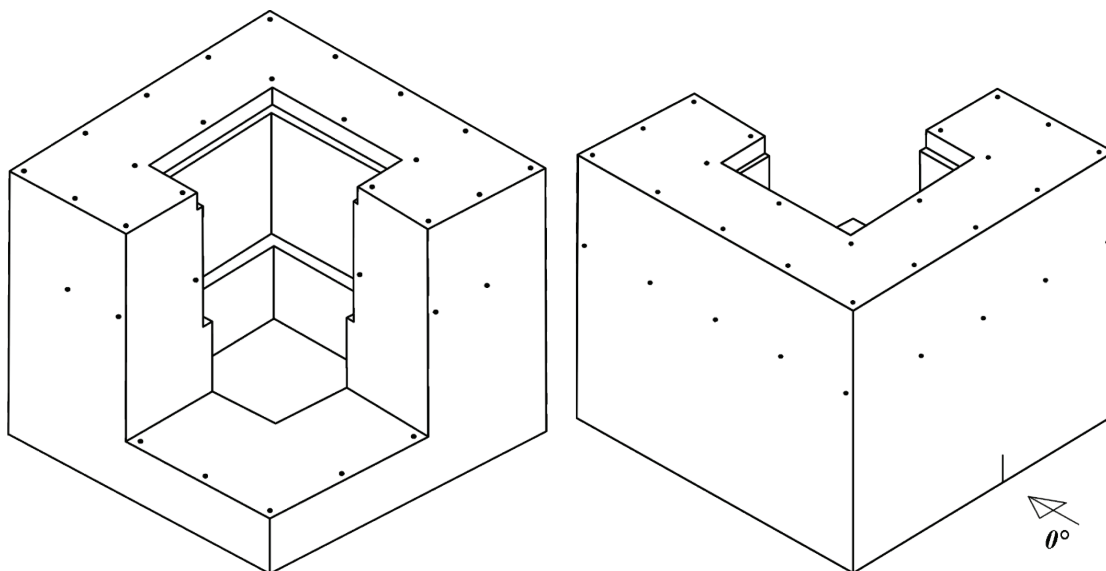


Fig. 4 Placement of pressure taps on the idealised physical model of the ruin

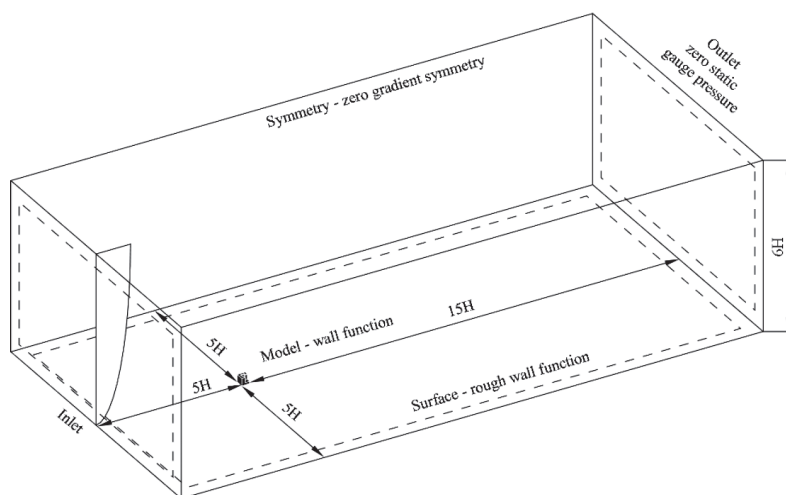


Fig. 5 Illustration of the computational domain's dimensions and boundary conditions

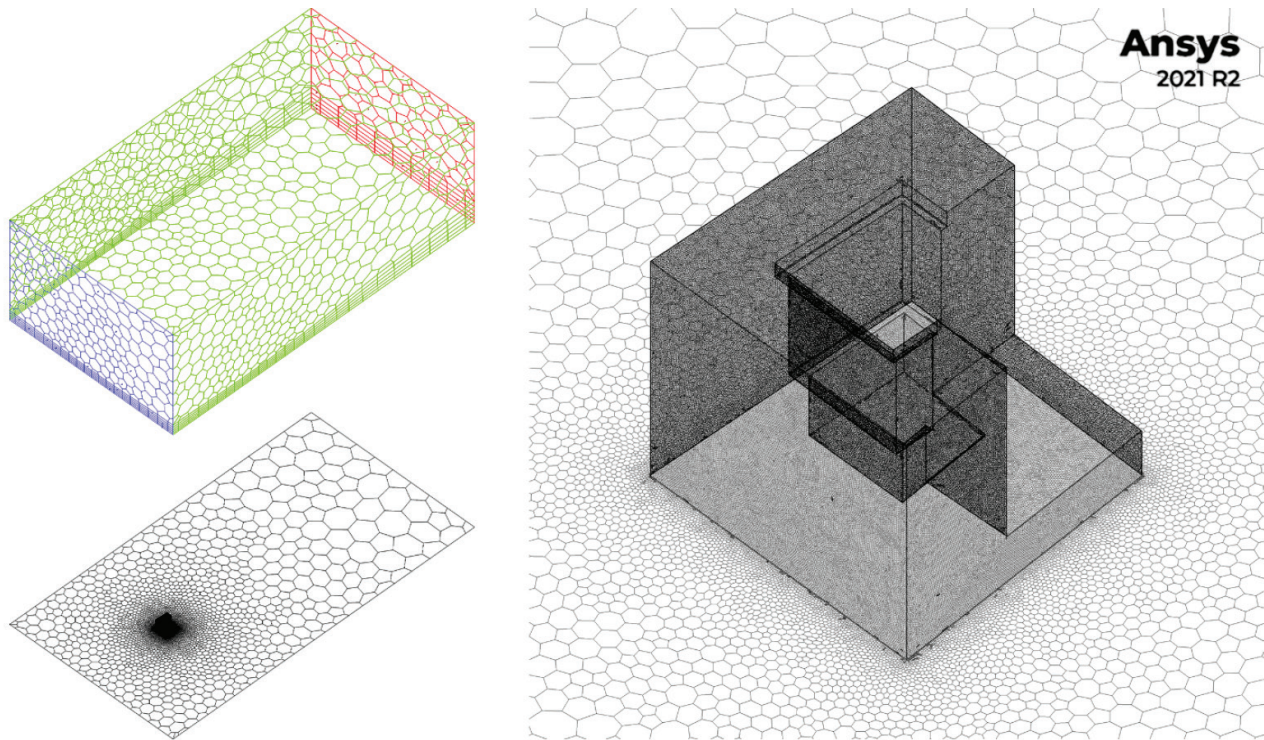


Fig. 6 Illustration of polyhedral mesh used; on the left, a blue inlet area, green symmetry area with a zero gradient, red outlet area, and a region of the terrain with a uniform roughness and with thickening mesh at the location of the idealised ruin model; on the right, details of the mesh on the surface of the idealised ruin model

The computational mesh was modelled in the meshing component of the ANSYS Workbench environment. The growth rate for the sizing and inflation values was changed from the basic value of 1.2 to 1.1 for the model. In this case, the mesh was automatically created using tetrahedrons. An element face-sizing condition with a value of 1 mm and an inflation condition for the first five mesh layers with a height limitation of 1 mm for the first layer were entered for the surface of the ruin model. For the first five mesh layers, a smooth transition condition was also applied to the inflation condition on the terrain surface. There were approximately 7.5 million cells and 1.8 million nodes in the resulting tetrahedral mesh. The quality indicators included the mean orthogonal quality of 0.77 and mean skewness of 0.23 (Mesh Quality, 2022; Fatchurrohman and Chia, 2017; 6.2.2 Mesh Quality, 2009). From there, the mesh was directly converted to a polyhedral in Fluent (Fig. 6).

A logarithmic profile of the mean wind speed $v_m(z)$ was set at the input area of the computational domain according to the experimental measurements in BLWT:

$$v_m(z) = \frac{\dot{v}}{\kappa} \ln \frac{z + z_0}{z_0} \quad (2)$$

where $v_m(z)$ is the mean wind speed at height z above the terrain [$\text{m}\cdot\text{s}^{-1}$]; $\dot{v}$ is the shear or friction velocity [$\text{m}\cdot\text{s}^{-1}$]; κ is the Von Kármán constant ($\kappa \approx 0.41$); z is the height above the terrain [m]; and z_0 is the length of the roughness with a value of 0.00179 m, which is also set as a terrain with a uniform length of roughness. The following equations were used to specify other parameters in the input and output domain boundaries:

$$k(z) = \frac{\dot{v}^2}{\sqrt{C_\mu}} \quad (3)$$

$$\varepsilon(z) = \frac{\dot{v}^3}{\kappa(z + z_0)} \quad (4)$$

$$\omega(z) = \frac{\varepsilon}{k} \quad (5)$$

where $k(z)$ is the turbulent kinetic energy [$\text{m}^2\cdot\text{s}^{-2}$]; C_μ is the constant of the k - ε model; ε is the dissipation rate of the turbulent kinetic energy [$\text{m}^2\cdot\text{s}^{-3}$]; and $\omega(z)$ is the specific dissipation of the turbulent kinetic energy [s^{-1}]. The sides and top of the computational domain were set as symmetrical boundaries with a zero gradient. The surface of the idealised ruin model had a roughness height set at 0.5 mm with a roughness constant of 0.5, while the terrain had a roughness height of 1.79 mm with a constant of 0.7.

A series of Reynolds-Averaged Navier-Stokes (RANS) turbulence models were used to simulate the eight wind directions in 45° steps, namely, Standard k - ε (sKE), Realizable k - ε (rKE), Re-Normalization Group k - ε (rngKE), Standard k - ω (sKO), Generalised k - ω (gekoKO), New Baseline k - ω (bslKO), and Shear Stress Transport k - ω (sstKO) (Menter et al., 2021; Menter, 1994; CFD Online, n.d.).

In order to further verify the computational model, several variants of the internal settings of the computational domain and mesh for one flow direction were applied, as shown in Tab. 1 and Fig. 7.

Tab. 1 Summary of the internal domain and mesh settings

	Mesh									k - ϵ Wall Treatment
	Sizing		Inflation 5 layers			Elements [mil]	Nodes [mil]	Quality		
	Growth Rate	Ruin Model	Growth Rate	Ruin Model	Surface			Ortho.	Skew.	
1	1.1	Face 1 mm	1.1	FLT 1 mm	Smooth Trans.	7.5	1.8	0.77	0.23	Stand.
2	1.1	Face 1 mm	1.1	x	x	9.2	1.6	0.78	0.22	Stand.
3	1.1	x	1.2	FLT 1.1 mm	FLT 3.6 mm	0.5	0.3	0.76	0.24	Stand.
4	1.1	x	1.2	FLT 1.1 mm	FLT 3.6 mm	0.5	0.3	0.76	0.24	Enhan.
5	1.1	Edge 1 mm	1.2	FLT 1.1 mm	FLT 3.6 mm	4.2	2.5	0.75	0.25	Enhan.
6	1.1	Edge 2 mm	1.2	FLT 1.1 mm	FLT 3.6 mm	1.9	1.1	0.75	0.25	Enhan.

Tab. 2 Statistical evaluation of the results calculated in CFD by various turbulence models, which considered eight flow directions; the most positive values are in black, the most negative in gray; note that all the FB values are negative, meaning that the CFD results are overestimated in comparison to the BLWT results.

		sKE	rngKE	rKE	sKW	gekoKW	bslKW	sstKW	I.V.
0°	FB	-0.245	-0.223	-0.260	-0.288	-0.248	-0.284	-0.183	0
	R	0.820	0.916	0.838	0.851	0.864	0.845	0.950	1
	FAC1.3	0.300	0.625	0.375	0.375	0.400	0.425	0.875	1
45°	FB	-0.169	-0.178	-0.169	-0.229	-0.187	-0.208	-0.113	0
	R	0.896	0.912	0.896	0.878	0.898	0.874	0.915	1
	FAC1.3	0.700	0.650	0.625	0.575	0.675	0.600	0.675	1
90°	FB	-0.152	-0.195	-0.180	-0.226	-0.200	-0.216	-0.153	0
	R	0.914	0.936	0.919	0.917	0.927	0.912	0.949	1
	FAC1.3	0.525	0.575	0.525	0.450	0.475	0.500	0.700	1
135°	FB	-0.106	-0.167	-0.172	-0.187	-0.172	-0.179	-0.126	0
	R	0.898	0.939	0.933	0.880	0.927	0.874	0.941	1
	FAC1.3	0.775	0.725	0.700	0.600	0.700	0.700	0.700	1
180°	FB	-0.148	-0.194	-0.182	-0.224	-0.200	-0.216	-0.151	0
	R	0.872	0.931	0.891	0.888	0.898	0.862	0.931	1
	FAC1.3	0.575	0.550	0.500	0.475	0.550	0.475	0.675	1
225°	FB	-0.108	-0.143	-0.127	-0.205	-0.151	-0.174	-0.084	0
	R	0.911	0.940	0.913	0.878	0.911	0.875	0.930	1
	FAC1.3	0.725	0.675	0.750	0.575	0.750	0.575	0.800	1
270°	FB	-0.208	-0.216	-0.237	-0.270	-0.226	-0.254	-0.176	0
	R	0.804	0.897	0.816	0.849	0.841	0.844	0.923	1
	FAC1.3	0.375	0.525	0.300	0.325	0.375	0.325	0.725	1
315°	FB	-0.191	-0.248	-0.255	-0.287	-0.243	-0.239	-0.195	0
	R	0.894	0.926	0.903	0.863	0.898	0.862	0.919	1
	FAC1.3	0.800	0.675	0.625	0.475	0.650	0.625	0.725	1

2.4 Assessment of results and method of comparison

Three statistical methods were applied to assess the results: fractional bias (FB; ideal value (I.V.) = 0.0), correlation coefficient (R, I.V. = 1.0), and part of the results within factor 1.3 (FAC1.3, IV = 1.0) based on the following equations:

$$FB = 2 \frac{\bar{X} - \bar{Y}}{\bar{X} + \bar{Y}} \quad (6)$$

$$R = \frac{cov(X, Y)}{\sigma_X \sigma_Y} \quad (7)$$

$$FAC1.3 = \frac{1}{n} \sum_{i=1}^n N_i, \text{ where } N_i = \begin{cases} 1 & \text{for } \frac{1}{1.3} \leq \frac{X_i}{Y_i} \leq 1.3 \\ 0 & \text{for others} \end{cases} \quad (8)$$

where $\bar{X}$ is the average of the c_{pe} values measured in BLWT; X is the c_{pe} value measured in BLWT; $\bar{Y}$ is the average c_{pe} value calculated in CFD; Y is the c_{pe} value calculated in CFD; cov is the covariance; and σ_X, σ_Y are the standard deviations for X and Y (Franeek et al., 2021).

3 RESULTS

The experimental results from BLWT were compared with the CFD results calculated using seven turbulence models,

which considered eight directions of flow from 0° to 315° in 45° steps (Tab. 2).

The results from the experimental measurements in BLWT were subsequently compared with those calculated in CFD using variant internal computational domain and mesh settings and analysed using the same statistical methods (Tab. 3).

Once the previous two steps were evaluated and the optimal turbulence model (sstKW), calculation domain, and mesh internal settings were selected (combination #1), the next step was to analyse the most critical flow directions. As a result of the c_{pe} values calculated for the entire surface of the idealised ruin model, the extreme values were assessed: the greatest and lowest values $c_{pe, max}$ and $c_{pe, min}$, which were followed by the average and median $c_{pe, mean}$ and $c_{pe, M}$ values (Tab. 4). The flow directions, which deviated by 22.5° on both sides, were supplemented near the critical directions estimated in the first round of the analysis (Tab. 5).

4 DISCUSSION

As shown in Tabs. 2 and 3, negative values were observed throughout the comparison of the BLWT and CFD data by using the FB statistical method, which indicates that the CFD results were overestimated.

Tab. 2 marks the most accurate sstKW turbulence model. However, for some flow directions, the positive results from the data comparison point to the sKE and rngKE calculation models. Compared to the others, the sstKW model did not pro-

Tab. 3 Statistical analysis of the results calculated in CFD by different computational models of turbulence, which considered six different combinations of the computational domain and mesh; note that all the FB values are negative, meaning that the CFD results are overestimated in comparison to the BLWT results.

		sKE	rngKE	rKE	sKW	gekoKW	bslKW	sstKW	I.V.
1	FB	-0.197	-0.159	-0.199	-0.257	-0.199	-0.256	-0.145	0
	R	0.829	0.930	0.857	0.856	0.875	0.848	0.951	1
	FAC1.3	0.313	0.781	0.469	0.438	0.469	0.406	0.875	1
2	FB	-0.196	-0.261	-0.231	-0.261	-0.236	-0.261	-0.224	0
	R	0.877	0.933	0.913	0.853	0.914	0.831	0.951	1
	FAC1.3	0.500	0.344	0.469	0.406	0.500	0.219	0.656	1
3	FB	-0.186	-0.236	-0.206	-0.255	-0.193	-0.256	-0.200	0
	R	0.795	0.895	0.825	0.832	0.823	0.829	0.939	1
	FAC1.3	0.219	0.375	0.250	0.063	0.188	0.094	0.625	1
4	FB	-0.187	-0.236	-0.207					0
	R	0.794	0.895	0.825					1
	FAC1.3	0.219	0.375	0.250					1
5	FB	-0.236	-0.254	-0.244	-0.224	-0.239	-0.227	-0.247	0
	R	0.842	0.938	0.849	0.856	0.880	0.846	0.947	1
	FAC1.3	0.219	0.438	0.250	0.156	0.250	0.125	0.344	1
6	FB	-0.218	-0.251	-0.233	-0.221	-0.235	-0.225	-0.219	0
	R	0.824	0.936	0.830	0.859	0.897	0.848	0.950	1
	FAC1.3	0.250	0.469	0.250	0.156	0.438	0.125	0.625	1

duce any negative results. From this point of view, the data from the sKW, sKE and rKE models appear to be the least accurate.

Tab. 3 represents the most accurate sstKW turbulence model in the first combination of the computational domain and internal mesh settings. Later combinations yielded more positive results only within their specific calculation models.

Tab. 4 denotes the three flow directions as critical: (1) 0° in terms of the average value and the median value of the external pressure coefficients $c_{pe,mean}$ and $c_{pe,M}$; (2) 225° in terms of the highest value captured of the maximum external pressure coefficient $c_{pe,max}$, the lowest value of the minimum external pressure coefficient $c_{pe,min}$, and also one of the highest values $c_{pe,mean}$ captured; and similarly, (3) the 315° direction captured the most

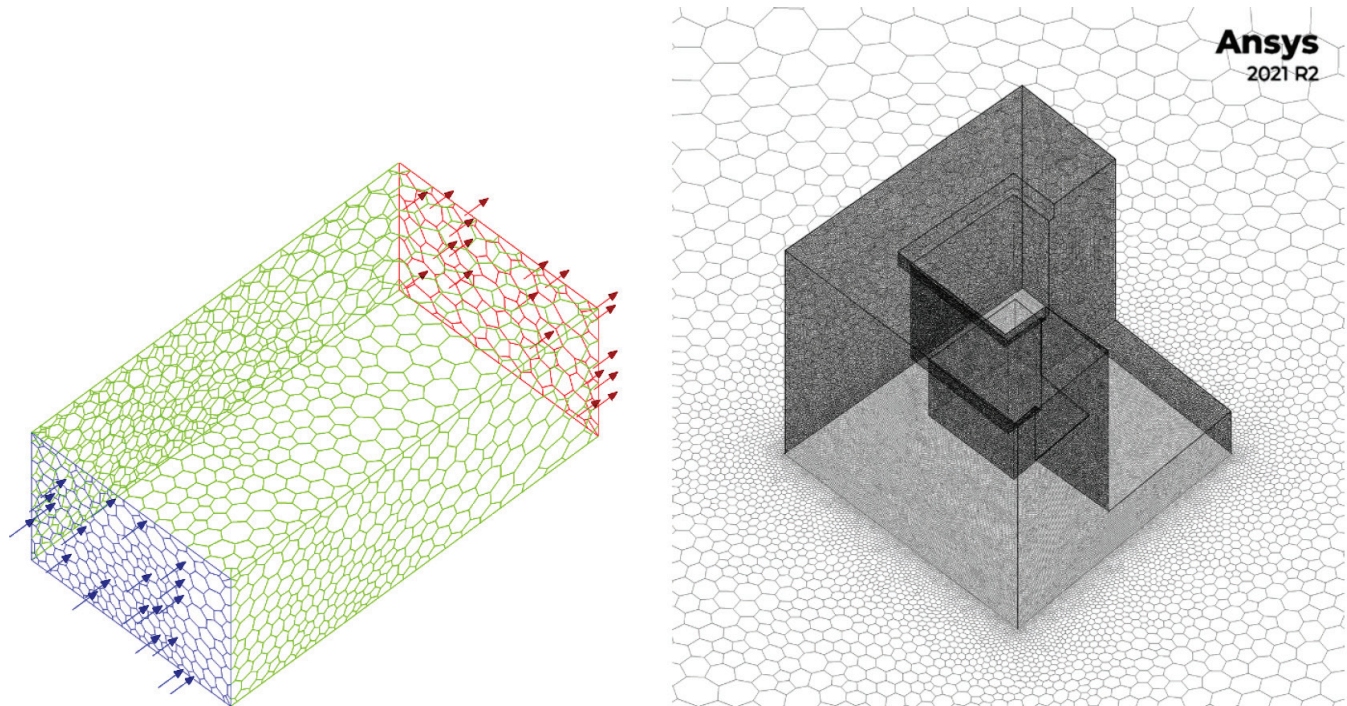


Fig. 7 Illustration of differences in the computation meshes considering the different variants of the settings; on the left, a view of the entire computational domain; on the right, a detail of the surface of the idealised ruin model; variant 2 from Tab. 3.

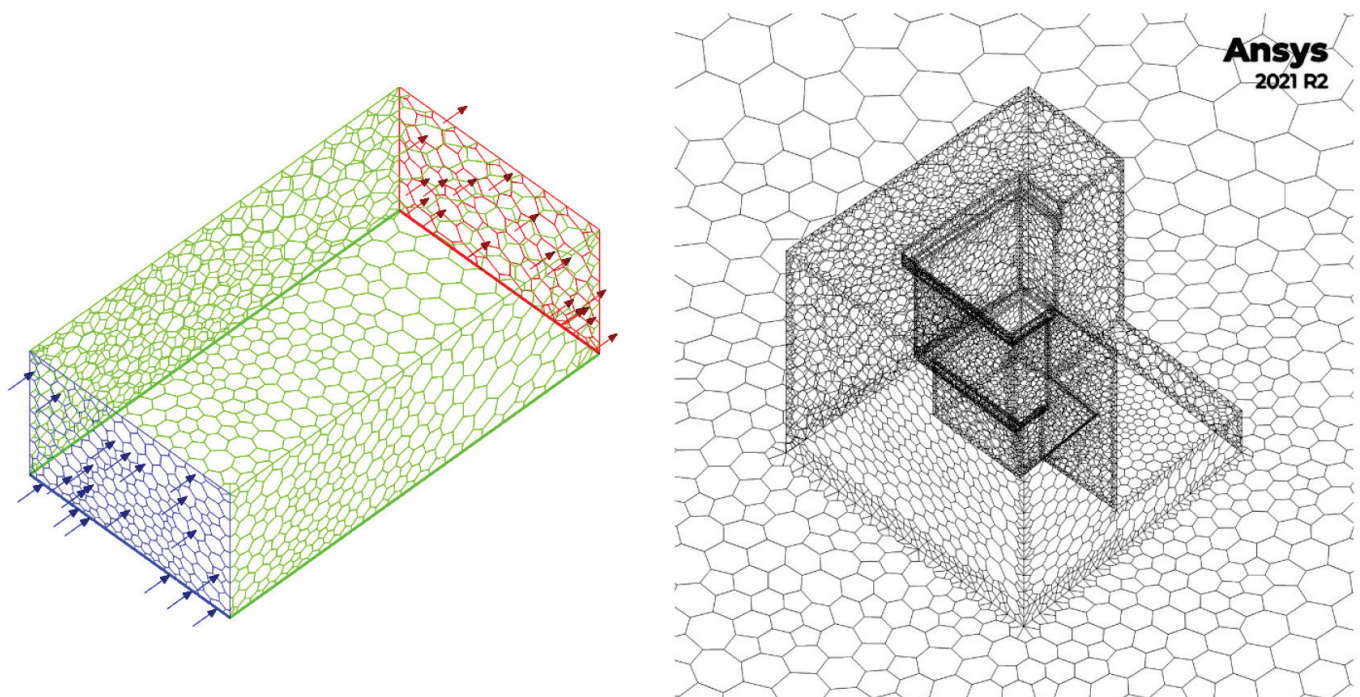


Fig. 8 Illustration of differences in the computation meshes considering the different variants of the settings; on the left, a view of the entire computational domain; on the right, a detail of the surface of the idealised ruin model; variants 3-4 from Tab. 3.

extreme values of $c_{pe,max}$, $c_{pe,min}$ and $c_{pe,M}$. However, some parameters monitored were directed at secondary flow directions. Therefore, the second analysis of the critical flow directions was supplemented with additional flow directions deviating by 22.5° on both sides of the three critical directions assessed previously.

The second analysis is summarized in Tab. 5. The 22.5° direction was evaluated as the most critical flow direction, be-

cause it was the only one that captured three extreme values of the four parameters monitored; and 247.5° because of the high values of the mean and median of the external pressure coefficients $c_{pe,mean}$ and $c_{pe,M}$. The second analysis confirmed the 315° flow direction as critical (Fig. 11).

In each of the critical flow directions $c_{pe,max}$ was monitored at the windward face of the model and the $c_{pe,min}$ value at the

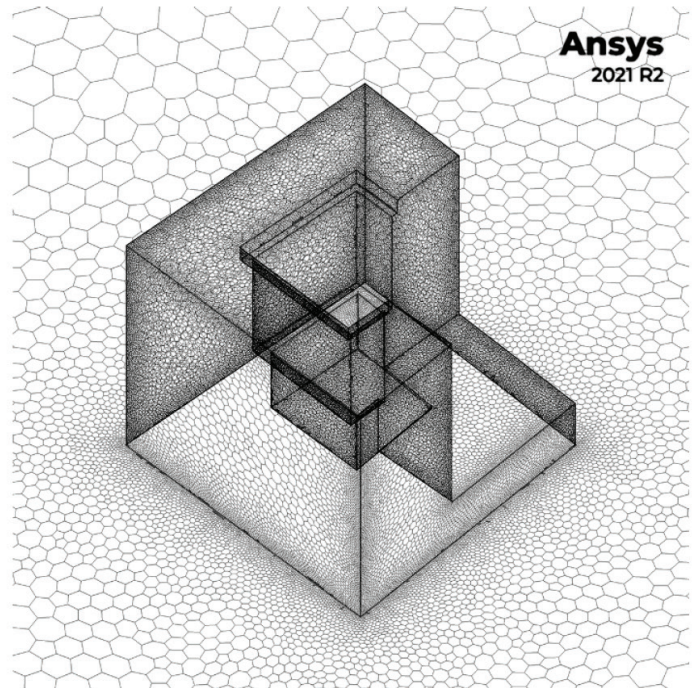
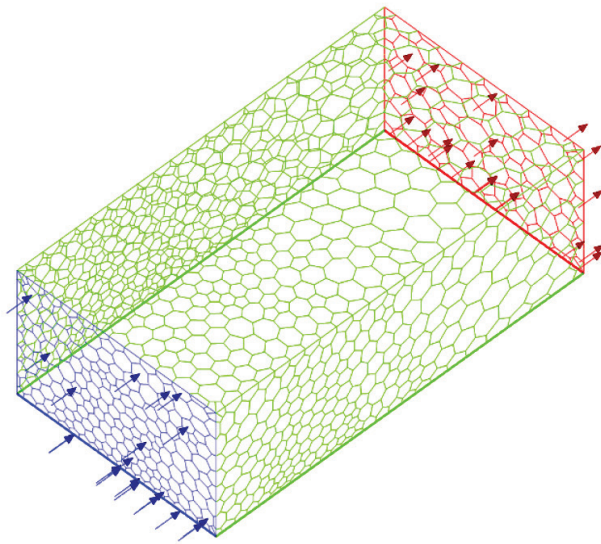


Fig. 9 Illustration of differences in the computation meshes considering the different variants of the settings; on the left, a view of the entire computational domain; on the right, a detail of the surface of the idealised ruin model; variant 5 from Tab. 3.

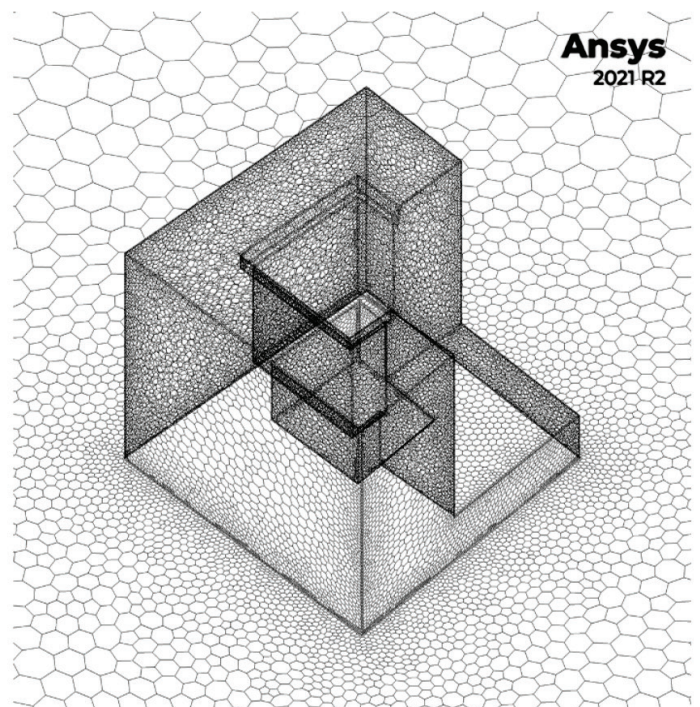
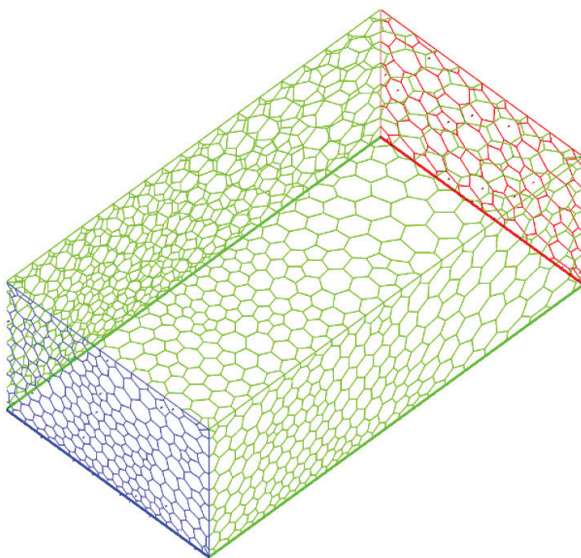


Fig. 10 Illustration of differences in the computation meshes considering the different variants of the settings; on the left, a view of the entire computational domain; on the right, a detail of the surface of the idealised ruin model; variant 6 from Tab. 3.

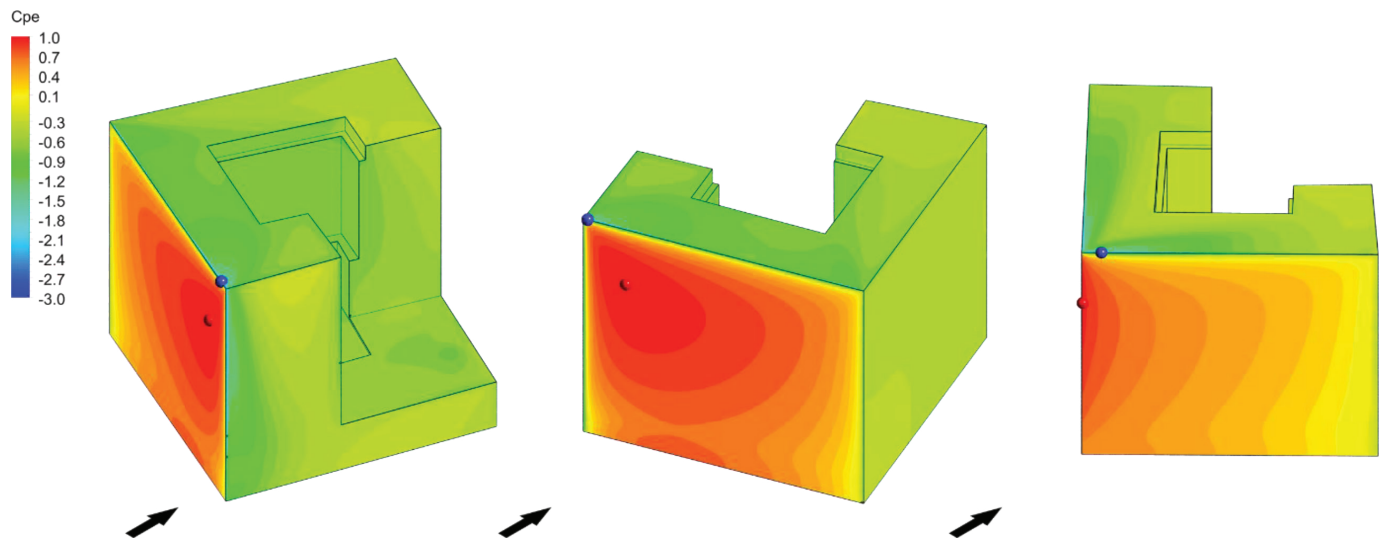


Fig. 11 Distribution of the external pressure coefficient c_{pe} illustration for flow directions 22.5° , 247.5° and 315° , with locations of $c_{pe,max}$ (red) and $c_{pe,min}$ (blue).

edge, where the windward face meets the head of the model's walls (Fig. 11).

4 CONCLUSION

Based on a statistical analysis of the values of the external pressure coefficient c_{pe} at 40 taps on the surface of the idealised ruin model, the sstKW turbulence model has been evaluated as the most appropriate model. These values were obtained by both experimental measurements in BLWT and numerical calculations in CFD during the simulation of eight flow directions. An additional evaluation of the combination of the computational mesh and the domain internal settings for one flow direction was conducted to verify the suitability of the sstKW turbulence model, based on a statistical analysis of the results. The critical flow directions of 22.5° , 247.5° and 315° were analysed and evaluated, based on the maximum, minimum, mean, and median values of the external pressure coefficient c_{pe} on the surface of the idealised ruin model, which was obtained by calculations in the optimized sstKW turbulence model.

Acknowledgements

As part of the Program for Support of Young Researchers at the Slovak University of Technology in Bratislava, the equipment for the construction of a idealised physical ruin model was purchased through the project *Simulation of the impact of the shape of the roof over the ruin on the distribution of wind pressure in critical areas* (BOREAS). A physical model of the ruin was created on the 3D printers at the Department of Architecture of the Faculty of Architecture of the Slovak University of Technology in Bratislava, which was acquired with the support of the *Penta Foundation*.

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A STUDY OF THE SEDIMENTATION AND STORAGE CAPACITY DEPLETION OF A RESERVOIR

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Abstract

Theoretical and empirical equations have previously been proposed to compute the sediment trap efficiency of a reservoir. The empirical equations were developed by analyzing a limited amount of field data for reservoirs in selected areas; thus, their suitability is restricted to reservoirs in regions with similar hydrological characteristics. In this study, a sediment trap-efficiency equation, which accounts for the dynamic behavior of sedimentation and the hydraulic conditions of a reservoir, was deduced from the principle of mass conservation. Using this equation, the approximate analytical expression for the storage capacity of the reservoir was obtained. Both equations were applied to investigate real-life problems involving the impacts of reservoir sedimentation, and their predicted solutions were compared with the field data and the results of the theoretical and empirical equations. The results of this investigation reveal that the proposed equations are capable of simulating the sediment trapping and storage capacity depletion characteristics of large reservoirs, which are a measure of the adverse effect of sedimentation on the performance of hydraulic structures. These equations can potentially be incorporated into a large-scale watershed model for estimating a sediment budget.

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Key words

- Trap efficiency,
- Reservoir sedimentation,
- Sediment yield,
- Blue Nile,
- Suspended sediment,
- Sediment-rating curve.

1 INTRODUCTION

Reservoir sedimentation is the gradual process of the accumulation of sediment in a man-made lake. In many parts of the world, sedimentation causes serious problems concerning water management, flood control, and the production of energy. As noted by Auel and Boes (2012) and Schleiss et al. (2010), the worldwide mean annual loss of the storage capacity of reservoirs due to sedimentation is higher than the increase in storage volume from the construction of new reservoirs. It is estimated that between 0.5-1% of the total storage capacity is lost annually (White, 2019). Furthermore, the time rate of the sedimentation in reservoirs varies with their design and the magni-

tude of their sediment load. Generally, smaller reservoirs lose their gross capacity at higher rates (Heinemann, 1984; Wisser et al., 2013). The results of numerical models showed that 4 to 5 Gt of sediment is trapped by reservoirs annually (Vörösmarty et al., 2003) compared to the Anthropocene-based global sediment load of 12.8 Gt/y (Syvitski and Kettner, 2011). It is obvious that large increases or decreases in such loads can change the dynamic balance of river systems and delicate ecosystems.

A simplified theory of the trap efficiency of a sedimentation basin was first developed by Hazen (1904). He considered the gravitational settling of sediment particles under different hydraulic conditions and obtained an expression for the

trap efficiency based on residence time, settling velocity, and particle size. Since then, methods to predict reservoir sedimentation have been the theme of several empirical studies. These studies considered trap efficiency as the most informative descriptor of reservoir sedimentation. The trap efficiency of a reservoir depends on many factors such as the capacity and geometric shape of a reservoir, the inflow discharges of sediment and water, the method of operation, and the settling velocity (Brune, 1953; Morris and Fan, 1998; Verstraeten and Poesen, 2000; Xiaoqing, 2003). Many empirical equations have been proposed, as part of the solutions to complex sedimentation problems, to predict the trap efficiency of reservoirs in temperate environments (Espinosa-Villegas and Schnoor, 2009; Heinemann, 1984; Verstraeten and Poesen, 2000). Brown (1943) was the earliest researcher to develop a curve that shows the relationship between trap efficiency and the ratio of the reservoir capacity to a watershed drainage area. A relatively better approach was later presented by Churchill (1948), who employed the ratio of the period of retention to the mean flow velocity through the reservoir as the sedimentation index and related it to the trap efficiency. To some extent, Churchill's approach considered the flow dynamics of a reservoir. For small dry or semi-dry reservoirs with bottom outlets, however, it is difficult to estimate the Churchill sedimentation index as these outlets only operate during infrequent short-lasting events (Dendy, 1974). By using the data of Churchill and those added by Borland (1971), Bube and Trimble (1986) modified the Churchill curves by applying a smoothing spline procedure. As noted by Brune (1953), reservoirs with the same watershed area could have different inflow characteristics and hence different trap efficiencies. Alternatively, he proposed the ratio of the reservoir capacity to the annual inflow and correlated it to the trap efficiency of normal ponded reservoirs. The two different parameters used by Brown and Brune provide only a limited description of the watershed hydrology and reservoir geometry; they are independent of the sediment characteristics. Compared to Brune's approach, the more descriptive Churchill method may provide a better result. The trap-efficiency curves of Brune were further investigated by Dendy (1974), Gill (1979), Heinemann (1981), Lewis et al. (2013), and Siyam et al. (2001). They developed empirical and semi-theoretical equations, which reasonably represented the curves. However, the weighting factor used by Lewis et al. (2013) for estimating the overall trap efficiency was not derived from the daily volumes of sediment transported into the reservoir. The application of the empirical methods described above is restricted to reservoirs in regions with similar hydrological conditions, as noted by Verstraeten and Poesen (2000).

Theoretical models based on the mechanics of sediment deposition were also applied to predict reservoir sediment trap efficiency. Camp (1946) developed a theoretical model based on the simplifying assumptions of a quiescent flow, a constant concentration of suspended particles at a vertical section near the inlet end, and no resuspension of any settled particles. For highly turbulent open-channel flow conditions, Chen (1975) extended Camp's model. He further investigated the applicability of the extended model and compared his results for turbulent flows with the results of the empirical methods of Brune and Churchill. Karaushev (1966) derived an analytical equation for predicting the trap efficiency of small reservoirs. He accounted

for the effects of the hydraulic characteristics of a flow and the sediment transported. A comparison of his equation with the Brune trap-efficiency curves clearly reveals the inadequacy of Brune's empirical generalization. For a reservoir having a small annual inflow relative to its storage volume, however, the accuracy of Karaushev's equation is questionable. Following Camp's approach, Curtis and McCuen (1977) developed a theoretical model based on the sedimentation of a small idealized settling tank. They used the model to examine the effects of the distribution of the sediment sizes, initial basin storage, outflow discharge, and basin depth on the trap efficiency of a fictitious pond. An overview of theoretically-based trap-efficiency models for variable discharge conditions was provided by Verstraeten and Poesen (2000).

Numerical approaches have also been used for a detailed analysis of flow-sediment interactions in a reservoir. Dhamotharan et al. (1981) developed a one-dimensional (1D) unsteady convective-diffusive transport model to predict the vertical concentration profile and deposition rate of suspended sediment. Their model is capable of computing the trap efficiency as a function of a reservoir's Peclet number and time. Toniolo et al. (2007) systematically examined the effects of turbidity currents on trapping efficiency using a simplified 1D numerical model that captures the dynamics of an internal muddy pond. By using soft computing data-driven techniques, Garg and Jothiprakash (2010) estimated the trap efficiency of a reservoir. Using a similar approach, Parsaie et al. (2018) also computed the trap efficiency of a retention dam. The results of both studies were verified with the measurements. Compared to empirical methods, numerical models need a large volume of data for their calibration and validation. This raises a practical question about the extent of evaluating the tasks of a model's performance (see Bennett et al., 2013). For a newly constructed reservoir, however, the data necessary for the model's construction are not available. Hence, one could prefer empirical or theoretical methods as a means of studying the effects of reservoir sedimentation, thus providing insight into the dynamics of watersheds.

The current study is aimed at developing a reservoir sediment trap-efficiency equation to overcome the aforementioned drawbacks. The proposed equation is capable of describing the dynamics of reservoir sedimentation and does not require a large volume of suspended sediment concentrations and flow rate data for calibration. This equation is applied to investigate different cases of reservoir sedimentation, and its performance is assessed by comparing its results with the field data and the results of theoretical and empirical methods. It is apparent that accurate quantification of sediment trapping in reservoirs leads to a reasonable prediction of storage capacity losses. Using the trap efficiency equation, an approximate analytical expression for estimating such losses is proposed. Besides, a set of design charts is presented to assist an evaluation of the remaining capacity of the reservoir. The prediction of the sediment trapping and storage capacity depletion characteristics of a reservoir using such equations allows practicing engineers to plan the daily operation and maintenance of a reservoir through dredging or sluicing operations. In the following sections, the derivation of the equations is briefly discussed.

2 MODELING TRAP EFFICIENCY

The sediment mass-balance equation for a reservoir can be expressed as

$$V \frac{dC}{dt} = C_i Q - CQ - w_s A_s C, \quad (1)$$

where V is the volume of the reservoir; C_i is the depth-averaged inflow sediment concentration; Q is the discharge; C is the depth-averaged concentration of sediment in the reservoir; w_s is the settling velocity of sediment; t is the time; and A_s is the surface area of the reservoir. The above equation assumes that the sediment in the reservoir does not interact with the surrounding environment. After dividing both sides of the equation by V and rearranging the terms, the following expression is obtained:

$$\frac{dC}{dt} + \Gamma_0 C = \frac{\Gamma_1}{V}, \quad (2)$$

$$\Gamma_0 = \frac{Q + w_s A_s}{V} = \frac{Q}{V} + \frac{w_s}{H_R}, \quad (3)$$

$$\Gamma_1 = C_i Q, \quad (4)$$

where $H_R (= V/A_s)$ is the mean depth of the reservoir. For a quasi-steady flow situation, the inflow discharge can be described by a step function. Using this approximation and imposing an initial condition of $C = 0$ at $t = 0$, Eq. (2) is integrated by applying an integrating factor method (Polyanin and Manzhirov, 2007), which results in

$$C = \frac{\Gamma_1}{\Gamma_0 V} (1 - \exp(-\Gamma_0 t)). \quad (5)$$

The trap efficiency of the reservoir represents the fraction of the incoming sediment that is retained in a reservoir and is expressed as

$$\eta = \frac{q_{si} - q_{so}}{q_{si}}, \quad (6)$$

where q_{si} and q_{so} are the inflow and outflow sediment discharges, respectively, and η is the reservoir sediment trap efficiency. Substituting Eq. (5) into Eq. (6) and simplifying the resulting expression gives

$$\eta = 1 - \left(\frac{1 - \exp\left(\frac{-1 - \Omega\sigma}{\Omega}\right)}{1 + \Omega\sigma} \right), \quad (7)$$

$$\Omega = \frac{V}{Qt} = \frac{V}{I}, \quad (8)$$

$$\sigma = \frac{w_s I}{H_R}, \quad (9)$$

where I is the inflow volume; Ω is the ratio of the reservoir capacity to the inflow volume; and σ is a sedimentation parameter, which is essentially governed by the dynamics of sedimentation in the reservoir. The above equation can be applied to estimate the trap efficiency of a reservoir with a shorter hydraulic residence time due to high intra-annual flow variability. As reported by Dey et al. (2019), several theoretical and empirical equations are available to estimate the settling velocity of natural sediment particles. In this study, a simple but accurate settling velocity equation, which was proposed by Cheng (1997a,b), is employed.

3 RESERVOIR CAPACITY EQUATION

The life of a reservoir primarily depends on its storage capacity loss caused by sediment deposition. The change in a reservoir's capacity over time due to sedimentation is given by

$$\frac{dV}{dt} = -\frac{q_{si}}{\gamma} \eta, \quad (10)$$

where $\bar{\gamma}$ is the time-averaged sediment density. The variation of the sediment density over time is approximated by the Lane and Koelzer (1943) equation as follows:

$$\gamma(t) = \gamma_1 + K \log(t) = \gamma_1 + 0.4343 K \ln(t), \quad (11)$$

where $\gamma(t)$ is the density of the sediment; γ_1 is the density of the sediment deposited at the end of the first year of filling the reservoir; and K is the compaction factor. The recommended values of γ_1 and K for various reservoir operational conditions can be found in Lane and Koelzer (1943) and USBR (2006). Following the Miller (1953) approach, the time-averaged sediment density at the end of the operating period T becomes

$$\bar{\gamma} = \left(\frac{1}{T-1} \right) \int_1^T \gamma(t) dt = \gamma_1 + 0.4343 K \left(\frac{T}{T-1} \ln(T) - 1 \right). \quad (12)$$

The above equation is used here to simplify the integration of Eq. (10). Inserting Eq. (7) into Eq. (10) and rearranging similar terms together gives

$$\frac{dV}{dt} = -\frac{q_{si}}{\gamma} \left(\frac{(\Gamma_3 - 1) \exp\left(\frac{\Gamma_3}{\Omega}\right) + 1}{\Gamma_3 \exp\left(\frac{\Gamma_3}{\Omega}\right)} \right), \quad (13)$$

$$\Gamma_3 = 1 + \Gamma_2 = 1 + \Omega\sigma. \quad (14)$$

The value of Γ_3 depends on the nature of the reservoir sedimentation problem. For the case of low sediment gravitational settling, $\Gamma_2 \rightarrow 0$ and hence $\Gamma_3 \cong 1$. The exponential expression on the right-hand side of Eq. (13) is simplified by using the second-order expansion of the Maclaurin series (Polyanin and Manzhirov, 2007) as follows:

$$\frac{dV}{dt} = -\frac{q_{si}}{\gamma} \left(\frac{\Gamma_3 V^2 + I \Gamma_3^2 V + 0.5 I^2 \Gamma_3^3}{(\Gamma_3 - 1)(V^2 + I \Gamma_3 V + 0.5 I^2 \Gamma_3^2) + V^2} \right)^{-1}. \quad (15)$$

Integrating the above equation from T_0 to T and substituting the conditions that at $t = T_0$, $V = V_0$ and at $t = T$, $V = V$, the following equation is obtained:

$$V - V_0 + \frac{I}{2} \left(\ln \left(\frac{V^2 + (\Gamma_3 - 1)(IV + 0.5I^2\Gamma_3)}{V_0^2 + (\Gamma_3 - 1)(IV_0 + 0.5I^2\Gamma_3)} \right) \right) = -\frac{q_{si}}{\gamma} (T - T_0). \quad (16)$$

Dividing both sides of Eq. (16) by $-V_0$ yields non-dimensional expressions as follows:

$$1 - V_R - \frac{1}{2\beta} \left(\ln \left(\frac{2\beta^2 V_R^2 + 2\beta(\Gamma_3 - 1)V_R + \Gamma_3(\Gamma_3 - 1)}{2\beta^2 + 2\beta(\Gamma_3 - 1) + \Gamma_3(\Gamma_3 - 1)} \right) \right) = \Psi, \quad (17)$$

$$\Psi = \frac{q_{si}}{V_0 \gamma} (T - T_0), \quad (18)$$

$$\Gamma_3 = 1 + \Omega\sigma = 1 + \sigma\beta V_R, \quad (19)$$

where V_R is the ratio of the reservoir capacity at any time T to the original volume, and β is the ratio of the original volume to the inflow volume. Eq. (17) is a generalized expression for estimating the remaining capacity of the reservoir after T years of operation. For a given sediment inflow discharge, initial reservoir volume, and time-averaged sediment density, Eq. (18) is used to predict the dimensionless sediment discharge parameter Ψ . Using this value, Eq. (17) is solved iteratively for the normalized remaining capacity of the reservoir. In the next section, the suitability of this equation for reservoir sedimentation problems will be examined.

4 APPLICATIONS

In this section, the results of the application of Eqs. (7) and (17) to real-life problems are presented and discussed. To establish an appropriate level of confidence in their performance, Eq. (7) will first be calibrated, and then both equations will be validated by comparing their results with the field data for the sediment trap efficiency and storage capacity of a reservoir. Reservoirs with different storage capacities and inflow volumes from various river basins are considered to evaluate the predictions of these equations. These reservoirs are situated in regions having different hydrological characteristics. Additionally, the impact of sedimentation on the reservoir capacity of the Grand Ethiopian Renaissance Dam is investigated by considering two long-term sedimentation scenarios.

4.1 Sediment Trap Efficiency

Equation (7) was calibrated using the sediment trap-efficiency data of Brune (1953) and Kira (1978). Fig. 1 shows the results of the calibration. As discussed before, σ is not a universal physical constant. It depends on the hydraulic and sediment characteristics of the reservoir. Using a non-linear optimization technique which utilizes the generalized reduced gradient method (Lasdon et al., 1974), the errors between the predicted trap efficiencies and the measured data for a normal ponded reservoir were minimized. The coefficient of determination (R^2) and the root mean square error for the best-fit curve ($\sigma = 74.0$) were 0.93 and 7.62%, respectively. As the flow in a semi-dry reservoir is very turbulent, the Brune data for this reservoir were well fitted by Eq. (7) for $\sigma = 0.51$. In addition, the envelope around Brune's mean curve was described well by this equation with σ values of 19.9 and 136.9 for the lower and upper envelope curves, respectively.

The performance of Eq. (7) was further examined by comparing its results with the results of previous equations proposed by Karaushev (1966) and Gill (1979). The sediment trap-efficiency equation of Karaushev for small reservoirs reads as:

$$\eta = 1 - \left((1 - \Omega) \exp \left(\frac{-\Omega \varphi}{1 - \Omega} \right) \right), \quad (20)$$

$$\varphi = \frac{w_s T_s}{H_R}, \quad (21)$$

where T_s is the duration of the spill-over period in seconds. For primarily highly flocculated and coarse-grained sediments, Gill (1979) introduced the following trap-efficiency equation:

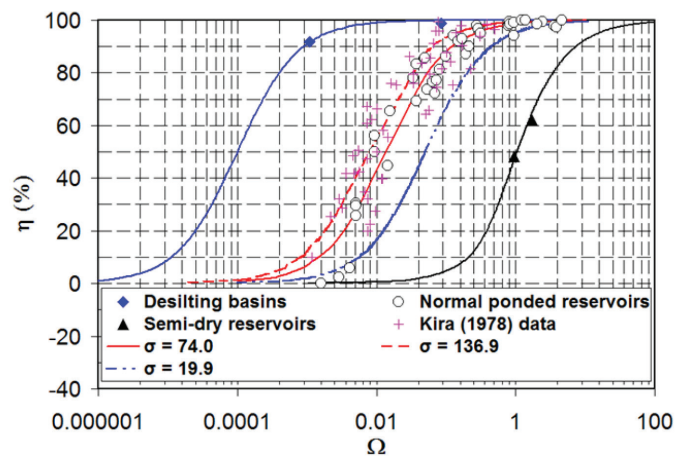


Fig. 1 Comparison of the calibration results of the proposed equation for reservoir trap efficiency with the field data of Brune (1953) and Kira (1978). The Brune data are depicted by three different geometric symbols, namely a diamond, circle, and triangle

$$\eta = \frac{\Omega^2}{0.9947 \Omega^2 + 62.97 \times 10^{-4} \Omega + 3 \times 10^{-6}}. \quad (22)$$

Figure 2 depicts a comparison of the results of these equations with the field data (Li et al., 2018; Tan et al., 2019). The data encompass the trap efficiency of small and large reservoirs in the major river basins of China. The active storage capacities of these reservoirs vary from 10 to 165000 Mm³. As can be seen, the results of Eq. (7) agree reasonably well with the measured data. Compared to the results of the Gill equation, the performance of the Karaushev equation is not satisfactory, especially for the higher values of the reservoir capacity to the inflow ratio ($0.025 < \Omega < 1.0$). The computed mean absolute relative errors for Eq. (7) and those of the Gill and Karaushev equations are 10.5, 11.8 and 15.8%, respectively. This implies that Eq. (7) is significantly more accurate than the Karaushev equation and is slightly more accurate than the Gill equation, in addition to providing a better description of the sediment trapping characteristics.

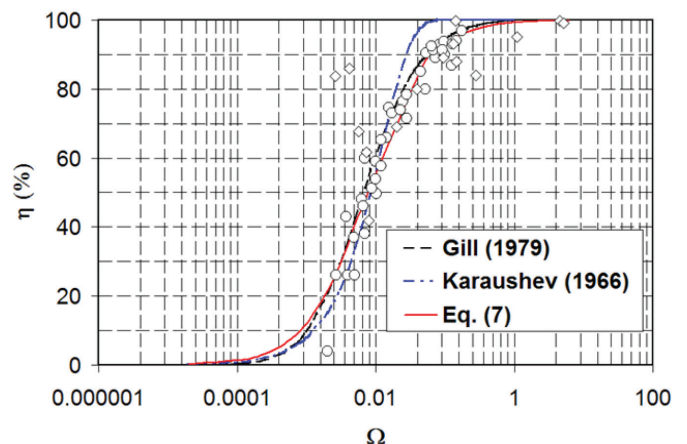


Fig. 2 Variation of the sediment trap efficiency η with the reservoir capacity to the inflow ratio Ω . The field data of Li et al. (2018) and Tan et al. (2019), which are indicated by the geometric symbols of a diamond and circle, respectively, were used to validate the numerical results

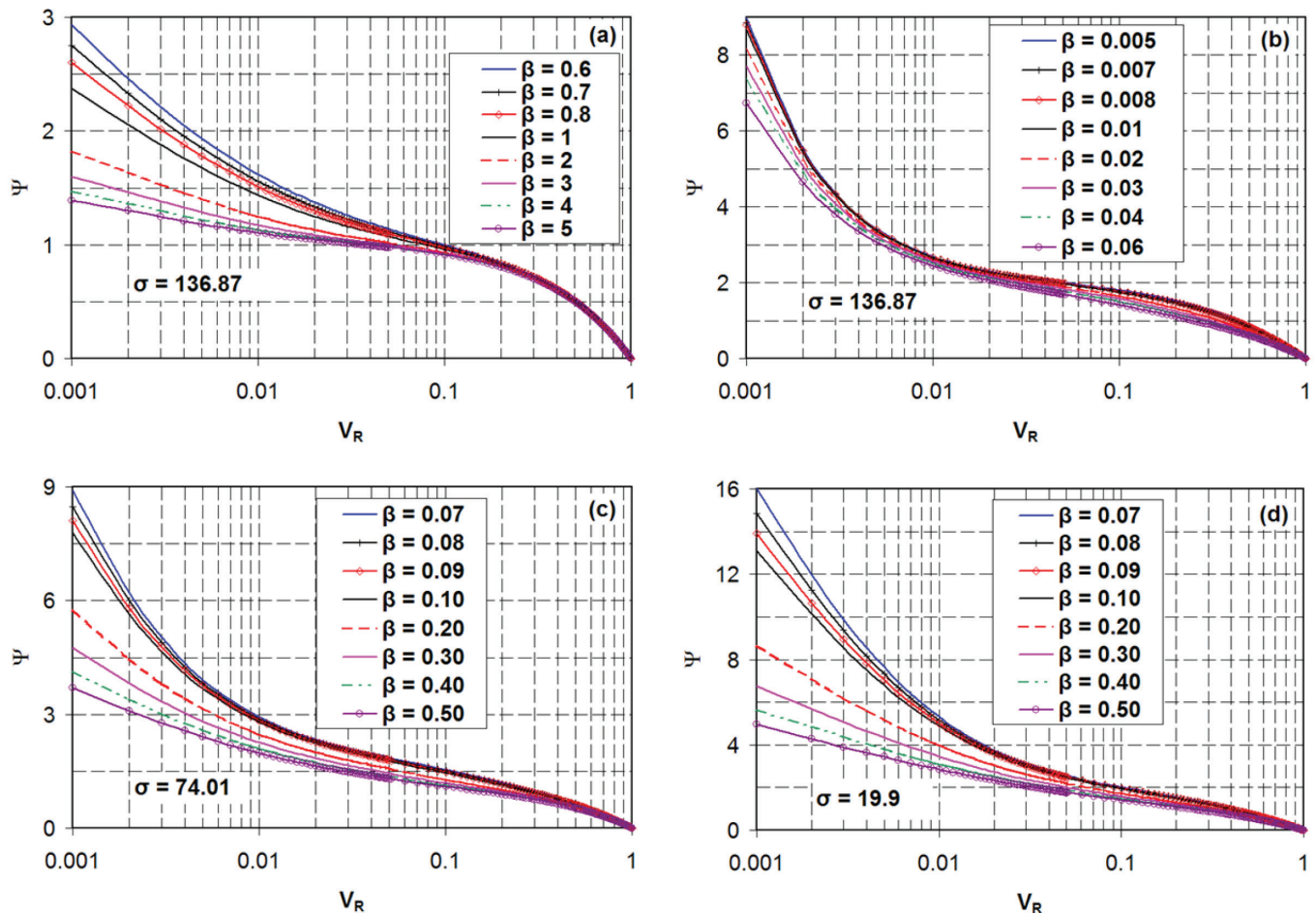


Fig. 3 Variations of the normalized total sediment inflow Ψ with the normalized remaining capacity of the reservoir V_R for various values of the initial reservoir capacity to the inflow volume ratio β . The calibrated values of σ were used to develop the charts

4.2 Storage Capacity of a Reservoir

4.2.1 Design charts for storage capacity prediction

Equation (17) is a non-linear algebraic equation and requires an iterative numerical procedure for its solution. The iterative procedure can be avoided by producing suitable charts that include the entire range of practical interest. The charts are prepared by plotting the normalized total sediment inflow rate Ψ against the dimensionless remaining capacity of the reservoir V_R for different values of σ and β . Fig. 3 shows the charts developed for typical values of β ranging from 0.005 to 5. For the long-term mean values of sediment discharge and inflow volume, the charts can be used to estimate the capacities of the reservoir after different periods of operation. Using a similar procedure, the useful life of a reservoir is easily predicted by considering various scenarios of reservoir sedimentation. For a given depletion volume of a reservoir, the chart can be used to determine Ψ , from which the required T can be computed.

4.2.2 Verification of the numerical results

The accuracy of Eq. (17) was examined by analyzing the storage capacities of six reservoirs in Kansas, USA, and comparing the predictions with the field data and the results of the Gill (1979, 1988) equation. The following empirical equation was proposed by Gill for fine-grained sediment:

$$1.0266(1 - V_R) + \frac{262.1 \times 10^{-4}}{\beta} \ln\left(\frac{1}{V_R}\right) + \frac{133 \times 10^{-6}}{\beta^2} \left(1 - \frac{1}{V_R}\right) - \frac{5.1 \times 10^{-7}}{\beta^3} \left(1 - \frac{1}{V_R^2}\right) = \frac{q_{st} T^{1-n}}{\gamma_1 V_0 (1-n)}, \quad (23)$$

where n is an exponent for the sediment density equation introduced by Gill (1988) and is given by

$$n = 0.5 \log\left(1 + \frac{2K}{\gamma_1}\right). \quad (24)$$

The selected reservoirs were built between 1948 and 1977, and their original capacities at the conservation-pool elevation vary from 34 to 524 Mm³. For some of these reservoirs, the volume-based sediment accumulation and yield values were estimated by analyzing bathymetry data (e.g., Kansas Biological Survey, 2011, 2014; Mau, 2001). In addition, continuous streamflow and turbidity data were collected at the gaging stations upstream and downstream from the reservoirs for estimating the total suspended sediment loads delivered to and released from the reservoirs as well as quantifying the capacity losses due to sedimentation (Juracek, 2011, 2013; Lee et al., 2008; Lee and Foster, 2013; Stone et al., 2015). In this study, the results of both methods were used to examine the performance of the proposed equation. Tab. 1 summarizes the results of the analysis. It is important to note that the dimensionless storage

Tab. 1 Comparison of the estimated dimensionless storage capacities with the measured data for selected reservoirs in Kansas, USA

Reservoir	Year built	V_0 (Mm ³)	Year of capacity survey	Measured V_R (%)	V_R (%) of Eq. (17)	V_R (%) of Eq. (23)
Cheney	1965	206	1998	96	97	97
Clinton	1977	159	2012	91	92	92
John Redmond	1964	126	2014	62	60	66
Kanopolis	1948	90	2010	66	70	70
Toronto	1960	34	2010	60	62	64
Tuttle Creek	1962	524	2010	57	57	57

capacities were estimated as of the years listed in Column 4 of Tab. 1. The dimensionless capacities of the reservoirs predicted by the two equations were slightly different, with mean absolute relative errors of 2.6 and 3.6% for Eqs. (17) and (23), respectively. This suggests that the accuracy of the proposed equation is acceptable for analyzing the impact of sedimentation on the capacity of the reservoirs after years of operation. The estimated annual loss rates of the reservoirs decrease as the size of the reservoir increases. A similar general trend for annual capacity loss has been noted by Crowder (1987).

4.3 The Grand Ethiopian Renaissance Dam

4.3.1 Description of the study area

The Grand Ethiopian Renaissance Dam (GERD) is currently under construction on the Upper Blue Nile (Abbay)

River approximately 750 km northwest of Addis Ababa and 20 km upstream of the Ethiopian–Sudanese border. The GERD is a roller-compacted concrete dam with a structural height of 145 m, and is complemented by a concrete-faced rockfill saddle dam approximately 5000 m long and 50 m high. The reservoir has a surface area of 1874 km² at a full supply level of 640 m above sea level (a.s.l.) and is classified as a very large reservoir according to Graf (2005). The total storage capacity of this single-purpose reservoir is 74 billion cubic meters (BCM) with an active storage volume of 59.2 BCM. Further details of the technical specifications of the dam can be found in IPoE (2013). The locations of the GERD and other proposed dam sites are shown in Fig. 4.

In the Upper Blue Nile River Basin (UBNRB), the predominant soil types are leptosols, alisols, nitosols, and vertisols. The texture of most of the soils consists of clay, clay loam, silt clay, and loam (BCEOM, 1999). Slopes in the basin range from as steep as 45% in the eastern part of the area

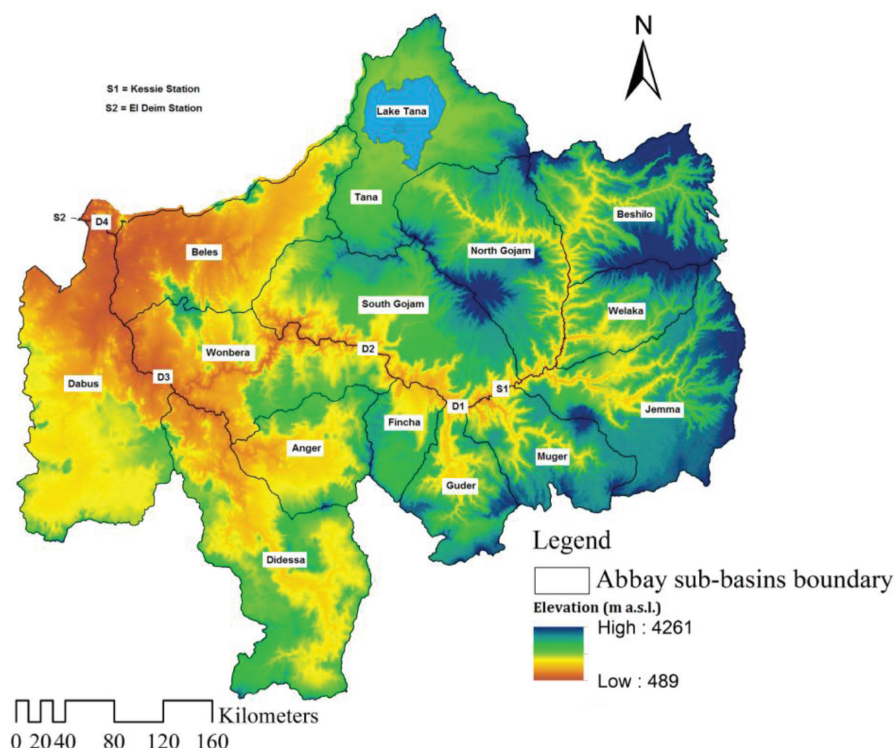


Fig. 4 The Upper Blue Nile River Basin and its sub-basins. The D1 to D4 symbols on the map indicate the locations of the Karadobi, Beko-Abo, Mandaya, and Grand Ethiopian Renaissance dams, respectively. The first three dams are proposed projects for hydropower generation

Tab. 2 Summary of the details of the selected streamflow gaging stations in the UBNRB

Station name and number	Location		Catchment area (km ²)	Percentage of the GERD catchment area
	Longitude (deg)	Latitude (deg)		
Anger (114002)	36.52	9.43	4,674	2.71
Didessa near Arjo (114001)	36.42	8.68	9,981	5.79
Guder (113005)	37.75	8.95	524	0.30
Tato (114010)	36.65	9.03	43	0.02
Teme at Motta (112030)	37.98	10.42	156.3	0.09
Blue Nile at Kessie (112001)	38.18	10.07	65,784	38.19
Blue Nile at Border (116002)	34.96	11.23	172,254	Downstream of GERD
El Deim (900012)	34.98	11.23	178,200	Downstream of GERD

to nearly level in Sudan with an average of about 4%. Land use is mostly agricultural with grasslands and croplands accounting for about 23 and 34% of the basin, respectively (BCEOM, 1998). Woodland, bushland, and shrubland cover 30% of the basin. Urban areas account for less than 1% of the basin.

The mean annual rainfall over the UBNRB varies from 800 to 2000 mm and increases with the altitude. The rainfall distribution is highly variable on both the spatial and temporal scales. Most of the annual precipitation is received during the wet season (June to September). Steep slopes and a lack of vegetative cover result in relatively high rates of sediment delivery to the main rivers. A recent study (Gebremicael et al., 2013) showed an increasing trend of soil erosion in the basin due to land-use/land-cover changes over the past 28 years.

4.3.2 Sediment-rating curves

Streamflow and suspended sediment data for the UBNRB were obtained from the Hydrology Department of the Ministry of Water, Irrigation, and Electricity, Ethiopia. The quality of the data from the selected gaging stations (see Tab. 2) was thoroughly examined, and some of the missing data were filled in with site-specific regression equations between neighboring stations. Because of the scarcity of long-term suspended sediment and bed-load data, the mean annual load was estimated by combining the short-term sediment-rating curve with a long-term flow-duration curve (Miller, 1951). A preliminary analysis of the streamflow and sediment concentration data at the Kessie gaging station indicated that a physically meaningful relationship between the two parameters could be obtained by developing sediment-rating curves for the wet and dry seasons of the water year. Accordingly, two logarithmically transformed data sets were developed and then analyzed using a linear regression approach. A statistical bias correction factor (Ferguson, 1986) was applied to improve the accuracy of the results. Fig. 5 depicts the suspended sediment concentration and sediment discharge rating curves. As expected, the sediment discharge relationships show significantly higher correlation coefficients than does the sediment concentration ($0.72 \leq R^2 \leq 0.98$). Additionally, flow-duration curves for a

30-year record (1973–2005) at this gaging station were developed, and sediment ratings were applied to them. For the intermediate drainage area between Kessie station and GERD, the average specific sediment yields of the sub-basins such as Beles, Dabus, Didessa, etc., were estimated using sediment concentration and daily streamflow (with a record length of 10 to 20 years) data from their gaging stations. Combining the contribution of these sub-basins and the sediment load at the Kessie station yields a mean annual suspended sediment inflow of 157.6 Mt/y at GERD. Due to the lack of measured data, the bed load can be estimated as a percentage of the suspended sediment load. The UBNRB is characterized by different topographic conditions, which range from flat plains to steep areas of the eastern part of the basin. Therefore, an allowance of 20% of the suspended sediment load was made for the bed-load component, giving a long-term mean annual total sediment load of 189.1 Mt. This is equivalent to 1098 t/km²/y when averaged over the total area of the dam catchment and is slightly higher than the instantaneous-sediment load measured at the El Deim station during the wet season of 2009 (Ali et al., 2014). As noted by Vanmaercke et al. (2012), sediment load estimates based on short measuring periods (less than 5 years) are more likely to underestimate the actual long-term mean annual load.

The current results were compared with the results of previous studies (e.g., Ali et al., 2014; Betrie et al., 2011; Easton et al., 2010; Gebremicael et al., 2013), which applied the Soil and Water Assessment Tool (SWAT) model to predict the annual sediment load from the UBNRB. These studies reported different results of the suspended sediment yield, varying from 117 to 147 Mt/y, at the basin outlet. The soil erosion rate is computed in the SWAT model using the Modified Universal Soil Loss Equation [MUSLE] (Williams, 1975). As an empirically-based equation, the MUSLE does not accurately simulate the soil erosion processes in the Ethiopian Highlands, where the contribution of sediment flux by gully erosion is significant. Consequently, the earlier studies underestimated the sediment yield. The trend of the sediment flux at the Kessie station was also examined, revealing that the sediment load was increased at an annual rate of 1.5% between 1989 and 2009. The results obtained are consistent with the trend analy-

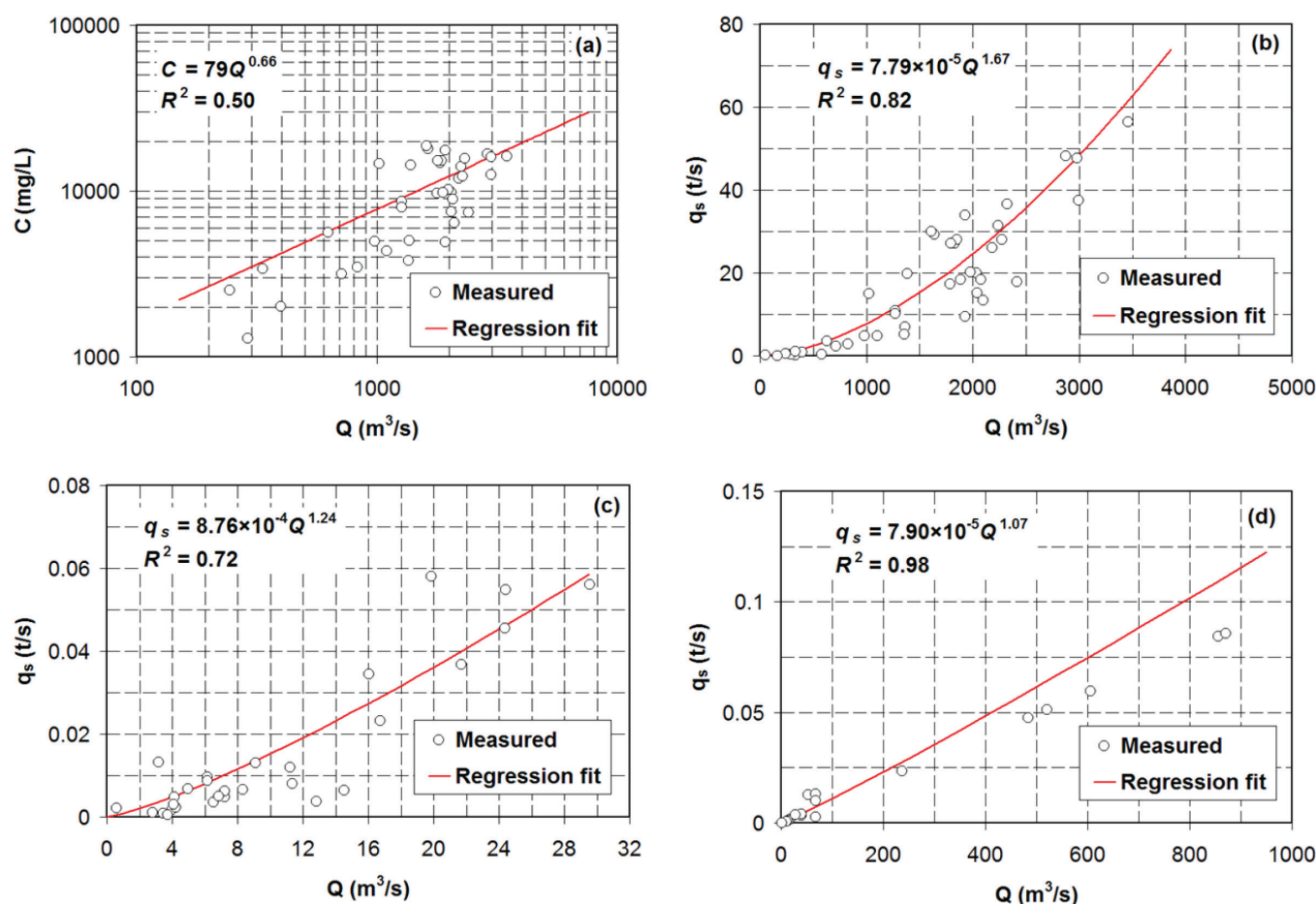


Fig. 5 Sediment concentration (a) and sediment discharge (b) rating curves for the wet season of the year at the Kessie gaging station on the Blue Nile River. The sediment discharge as a function of streamflow for the Teme and Guder Rivers is also shown in (c) and (d), respectively. The sediment discharge at a particular section of the river is denoted by q_s

sis results of Gebremicael et al. (2013).

4.3.3 Depletion of the reservoir storage capacity

Because of the longer record period of the available streamflow data and the greater frequency of measurements, the data at the El Deim station rather than those recorded at the Border gaging station were analyzed to compute the long-term mean annual inflow to the GERD reservoir. The computations resulted in a mean annual inflow of 48 BCM on the basis of the 1963–2010 streamflow records, which were obtained from the Ministry of Water Resources, Irrigation, and Electricity, Sudan. Using the computed mean annual sediment discharge and inflow rate, the storage capacities of the reservoir after various years of operation were estimated based on the following two long-term reservoir sedimentation scenarios:

1. Scenario 1: sedimentation of the reservoir is based on a constant mean annual inflow of sediment flux. This scenario assumes that the dam catchment area will not be subjected to significant land-use changes, so that its sediment yield remains unaltered.
2. Scenario 2: sedimentation of the reservoir is based on sediment discharge continuing to rise at 1.5% annually.

Examination of the field data demonstrated that the Blue Nile River transports significant quantities of very fine mate-

rial composed of silt and clay. Hence, the equations for fine-grained sediment were used to evaluate the remaining capacity of the reservoir. The results of the computation are summarized in Tab. 3. For both scenarios, the predictions of Eqs. (17) and (23) are almost similar. The storage capacity of the reservoir continuously decreases from the sediment load due to soil erosion resulting from land-cover changes. As can be seen, the reservoir loses 21 and 52% of its gross storage through sediment deposition in Scenarios 1 and 2, respectively, within 100 years. The initial trap efficiency of the reservoir was estimated at 96.9% and is expected to decrease with the age of the reservoir. The results of this

Tab. 3 Normalized storage capacities of the GERD reservoir after various years of operation

T (y)	Scenario 1		Scenario 2	
	Eq. (17) V_R (%)	Eq. (23) V_R (%)	Eq. (17) V_R (%)	Eq. (23) V_R (%)
25	94	94	92	92
50	89	89	81	80
75	84	84	66	65
100	79	79	48	47
150	69	69	13	2

study reveal that the reservoir loses its storage capacity at an average annual rate of 0.21% in Scenario 1. This rate is doubled in Scenario 2. In light of this sediment infill problem, a sediment management strategy, both upstream and within the reservoir, might be necessary for more sustainable energy production. Such a strategy should consider the sediment trapping effect of the proposed upstream dams on the GERD reservoir.

5 CONCLUSIONS

A sediment trap-efficiency equation, which aims to overcome the drawbacks of the existing methods, was developed based on the principle of mass conservation. The proposed equation considers the dynamic behavior of the sediment transported and the flow dynamics of the reservoir. Compared to a higher resolution numerical model, it does not require extensive field data for calibration and validation. Using this equation, the approximate analytical expression for predicting the storage capacities of the reservoir after various years of operation was obtained. These equations were applied to investigate real-life problems related to the impacts of reservoir sedimentation, and their results were compared with the field data and the results of theoretical and empirical methods.

The results of the analysis for the trap efficiency of the reservoir confirmed that the performance of Eq. (7) was satisfactory compared to the trap-efficiency equations of Gill (1979)

and Karaushev (1966). For a hydrologically small reservoir with $\Omega > 0.025$, the Karaushev equation overestimated the sediment trap efficiency. Furthermore, the suitability of Eq. (17) for predicting the loss of reservoir capacity due to sedimentation was examined. Compared to the reservoir capacity equation of Gill (1979,1988), the remaining capacity of the reservoirs was more accurately predicted by Eq. (17) with a mean absolute relative error of 2.6%. Overall, the results revealed that the accuracy of Eqs. (7) and (17) is acceptable for analyzing the sediment trapping and storage capacity depletion characteristics of large reservoirs.

As a test case, the impact of sedimentation on the reservoir capacity of the Grand Ethiopian Renaissance Dam was investigated based on two long-term sedimentation scenarios. Scenario 1 assumed a constant mean annual inflow of sediment flux, whereas Scenario 2 considered sedimentation based on a sediment discharge continuing to rise at 1.5% annually. The results demonstrated that the reservoir loses 21 and 52% of its gross storage through sediment deposition in Scenarios 1 and 2, respectively, within 100 years. In addition, Scenario 2 resulted in a mean annual storage capacity depletion rate of 0.45%, which is twice the resulting rate of Scenario 1. It is concluded that the equations developed can be applied to examine the impact of sediment accumulation on the life of a complex multi-purpose reservoir. Also, they can be coupled with a hydrological model for analyzing the sediment budget on a watershed scale.

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EXPERIMENTAL INVESTIGATION OF THE PERFORMANCE OF BURIED FLEXIBLE PIPE IN REINFORCED SAND

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Abstract

The experimental work of this study deals with the performance of pipe in sand with various responses subjected to incremental static loading where the backfill is reinforced with two types of reinforcement (geogrid and geocell) to investigate the effects of backfill cover. The pipes considered are installed in sand with different densities; i.e., loose, medium and dense sand, and a constant burial depth (2D, where D is the pipe diameter). It was concluded that the maximum deformation occurs at the crown of the pipe. The deformation increases when the sand density decreases. For a burial depth equal to 1.5 D, the maximum strain on the pipe crown increases by about 22.5% when the sand changes from dense to medium and 36.8% when it changes from dense to loose. The vertical stress above a pipe buried in loose sand is reduced by 8.3% when the geogrid width is 2B where B is the footing width, while it is equal to 22% when the geogrid width is B. When a geocell is used, the percent reduction is equal to 62%. The percent reduction in the vertical stress above a pipe buried in medium sand is about 28% when the geogrid width is 2B and 40% when the geogrid width is B and equal to 68% when the geocell is used.

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Key words

- Buried flexible pipe,
- Static loads,
- Soil reinforcement,
- Backfill density,
- Stress transfer,
- Surface settlement.

1 INTRODUCTION

Buried pipes have served to improve living standards since the beginning of civilization. Buried pipes serve in different applications such as drain pipes, sewer pipes, oil pipes, and subway tunnels (Moser and Falkman, 2008). Their actual behavior should be well understood in order to achieve the safety as one of the most important urban facilities (Moghaddas Tafreshi and Khalaj, 2008).

High-density polyethylene pipe has good potential for economical use in view of its inherent chemical and corrosion resistance, light weight, toughness, flexibility, easy splicing, and consequent easy handling (Goddard, 1995).

The analysis of buried plastic pipes is a problem involving soil-structure interaction. Plastic pipe has little inherent strength, so most of its ability to support a vertical load must be supplied by the passive pressure induced as the sides of the pipe move outward against the substantially vertical load, which is then carried by the surrounding soil by the mechanism of an arching action (Selig and Packard, 1987). This clearly means that the structural behavior of buried flexible plastic pipes is not only a property of the pipe, but is also a property of the earth surrounding the pipe.

The structural performance of buried plastic pipe is evaluated through deflection, ring bending, ring compression, strength, maximum strain, and buckling. The calculation and

determination of the structural performance is the essential assignment of the analysis and design of the buried plastic pipe. The deflection of buried plastic pipes is defined as the change in the vertical diameter of the pipe taken with respect to the neutral axis of the pipe wall.

Moghaddas Tafreshi and Khalaj (2008) assessed the behavior of small-diameter high-density polyethylene (HDPE) pipe (110 mm diameter and 4.03 mm wall thickness) buried in reinforced sand, which was then subjected to repeated loading (of an amplitude of 550 kPa). They examined the effect of between 1 to 5 layers of the reinforcement in the soil having relative densities of 42%, 57%, and 72%. The pipe was embedded at depths 1.5– 3 times its diameter. Testing was performed in a 550 mm wide trench. It was reported that the proportion of change in vertical pipe diameter and soil surface settlement can be reduced ultimately by about 40% and 51%, respectively, when using the stiffest reinforcement in the backfill of the highest density when the pipe is at its deepest embedment.

Mehrjardi and Moghaddas Tafreshi (2008) also developed a genetic algorithm linked together with a neural network (based on the test results of Tafreshi and Khalaj (2008) and determined that to achieve a 10 mm soil surface settlement and 2% pipe vertical diametric strain, one layer of a geogrid, a 75% soil relative density, and a ratio of 2.5 for the embedment depth of the pipe-to-pipe diameter would be required.

Although other researchers have studied the stress deformation and small strain shear-wave characteristics of rubber and particle mixtures (Lee et al., 2010) and the potential use of rubber as insulating backfill material for buried pipelines (Christ et al., 2010; Moghaddas Tafreshi and Dawson, 2012), there has been an absence of investigation into the protection of buried pipes with the use of geocell reinforcement and rubber waste.

Reinforcing soil with geosynthetics has been shown to be an effective method for improving the uplift capacity of granular soils. The pull-out resistance of the reinforcing elements is one of the most notable factors in increasing the uplift capacity. Mahdi and Katebia (2015) proposed a new reinforcing element, including elements (anchors) attached to an ordinary geogrid for increasing the pull-out resistance of the reinforcement. Thus, the reinforcement consists of the geogrid and anchors with cylindrical plastic elements attached to it, namely, grid-anchors. A three-dimensional numerical study, which employed the commercial finite difference software FLAC-3D, was performed to investigate the uplift capacity of pipelines buried in sand that were reinforced with this system. The models were used to investigate the effect of the pipe diameter, burial depth, soil density, number of reinforcement layers, width of the reinforcement layer, and the stiffness of the geogrid and anchors on the uplift resistance of the sandy soils. The outcomes revealed that, due to the development of a longer failure surface, the inclusion of a grid-anchor system in a soil deposit increases its uplift capacity to an outstanding degree.

Ahmed et al. (2015) presented the results of an experimental investigation that was conducted to measure the distribution of contact pressure on rigid pipe using tactile sensing technology. The method allows for a continuous pressure profile to be measured around the pipes using flexible sheets that can follow the cylindrical shape of the pipe. The physical model involved a buried pipe installed in granular material subjected to strip surface loading. The effect of introducing

a geogrid reinforcement layer above the pipe on the distribution of the contact pressure was also examined. The numerical framework was first validated using the experimental results and then used to investigate the detailed behavior of the soil-pipe system.

The finite element method was employed by Khademi-Zahedi (2019) to evaluate the behavior of buried medium density polyethylene (MDPE) pipes, which had been subjected to damage at the pipe crown. The stress analyses were performed using the ANSYS software finite element package. The stress distribution around the defect was determined under the aforementioned mechanical and thermal loading conditions. Then, the maximum values of the Von Mises stresses in the damaged, buried PE pipes were compared with their correspondingly reduced strength for a safe operation with a life expectancy of fifty years. The stress values increased with the following factors: an increase in the temperature, an increase in the diameter of the hole and a decrease in the elliptical hole diameter ratio (a/b).

Moghaddas Tafreshi et al. (2020) reported full scale experiments, under simulated heavy traffic, of geocell and expanded polystyrene (EPS) geofoam block inclusions to mitigate the pressure on, and deformation of, shallow buried, high density polyethylene (HDPE) flexible pipes, while limiting the surface settlement of a backfilled trench. A geocell of two pocket sizes and an EPS of different widths and thicknesses were used. The soil surface settlement, pipe deformation, and transferred pressure onto the pipe were evaluated under repeated loading. The results showed that using EPS may sometimes lead to larger surface settlements but can also alleviate pressure on the pipe and, consequentially, result in lower pipe deformations.

Three-dimensional finite element models were executed by Elshesheny et al. (2020) and validated to investigate the performance of buried flexible high-density polyethylene (HDPE) pipes, in unreinforced and multiple geogrid-reinforced sand beds, while varying the pipe burial depth, the number of geogrid-layers, and the magnitude of the cyclic loading applied. Soil-geogrid load transfer mechanisms due to interlocked soil in between the apertures of the geogrid-layer were modelled. In unreinforced and reinforced cases, increase in the depth of the pipe burial contributed to the decreasing deformations of the footing and pipe and the crown pressure until reaching an optimum value of the pipe burial's depth. On the contrary, the strain on the geogrid-layers increased with the increasing the depth of pipe burial.

Alotaibi et al. (2021) discussed the use of stiff three-dimensional geogrids to reinforce sandy soil bridging over existing buried polyvinyl chloride (PVC) pipe at a shallow depth subjected to strip static loading. Twenty-eight large-scale laboratory tests were conducted with variable geogrid geometries using single and double layers of geogrid reinforcement. In general, introducing the geogrid reinforcement reduced the stress transferred to the pipe crown below the center of the loading. For the given stress applied, the increase in the width of the geogrid has resulted in a reduction in the stress transferred to the pipe and the longitudinal strains developed in the pipe. Up to an 80% reduction in the longitudinal strains was observed in the buried PVC pipe as a result of the geogrid reinforcement.

Abdelfateh et al. (2021) presented a method for modeling the behavior of a pipeline based on the finite element analy-

sis by using PLAXIS 3D software, which was aimed at the determination of the pipe bending moment, the displacement over its length, and the evaluation of the vertical stresses in the soil under the pipe. A parametric study was carried out to investigate the effect of the depth of the pipe burial and the soil cohesion. It was found that, unlike laboratory models, the numerical analysis can account for the internal pressure in the pipe and the depth of the pipe burial. The finite-element analysis showed that the presence of fluid pressure inside the pipe results in a decrease in the maximum swelling of the soil by about 95%.

The present study is aimed at a determination of the effects of the mechanical parameters characterizing the soil-structure interaction developed in buried pipe installations in order to investigate the effects of using a geocell reinforcement on a buried pipe and evaluating this concept by laboratory scale modeling. The static loading has been investigated using a laboratory model to carry out a series of experimental tests for different parameters affecting the behavior of buried UPVC pipe. The pipes considered were installed in sand with different densities; i.e., loose, medium and dense sand, and a constant burial depth. Soil pressure cells and special data acquisition system were used for measuring stresses in the soil around the pipe and to measure the deformation of the pipe. The surface settlement was also measured by LVDT. The novelty comes from direct measurement of the stresses below the geocells to investigate any change in the stress transfer upon using these geocells.

2 EXPERIMENTAL WORK

In order to determine the required soil parameters, a soil-testing program was carried out. Routine soil tests were carried out to characterize the soil properties, namely, the grain size analysis, specific gravity, direct shear test, modified Proctor compaction test and the maximum and minimum dry density. The granular backfill materials used in this study were supplied from a site in the city of Karbala in Iraq. A relatively

well-graded sand was passed through sieve No. 10 (maximum size is 2 mm). The mechanical grain size analysis according to ASTM D422-07 was performed on the backfill material. The physical and mechanical properties of the sand are summarized in Table 1.

The tensile tests were conducted on PVC pipe that complies with BSI (2000) for underground sewer and drainage services. The pipe has an outer diameter of 110 mm and a wall thickness of 4 mm and, hence, a Standard Dimension Ratio (SDR) = $D/t = 27.5$. The length of the pipe was selected to be 750 mm. This pipe was subjected to tensile force, and the axial deformation was recorded. The tensile strength at a 10% axial strain of the pipe was measured to be 21 MPa and considered to be the ultimate tensile strength of the pipe material.

2.1 Description of experimental apparatus

A steel container was used to host the soil bed and pipe. The internal dimensions of the container were of a length of 1200 mm, a width of 1000 mm and a height of 1000 mm. One face of the steel had Plexiglas windows with 500 x 500 mm dimensions, which were used for monitoring the pipe deformation during the test. The soil box was used to study the behavior of the buried UPVC pipe as shown in Figure 1.

A steel loading frame was manufactured to support the piston. The stress and displacement patterns in the sand can be affected by the side boundaries of the soil container. In addition, due to the friction between the container walls and soil grains, the vertical stress in the sand can be reduced with the depth. To avoid the side friction of the walls, the ratio between the container's height and its diameter must be equal to or less than one (Garnier, 2002). The setup dimensions of the present study are within the influence zone found by previous investigators such as Springman (2007) and completely satisfy the influence zone determined from the theory of elasticity.

The axial load was applied through a hydraulic jack system as shown in Figure 1. The hydraulic jack frame had a

Tab. 1 Properties of the sand used

Index Property	Standards	Value
Specific gravity (Gs)	ASTM D-854	2.68
D_{10} (mm)	ASTM D 422	0.05
D_{30} (mm)	ASTM D 422	0.26
D_{60} (mm)	ASTM D 422	0.46
Coefficient of uniformity (Cu)	-	9.2
Coefficient of curvature (Cc)	-	2.94
Maximum dry unit weight (kN/m ³)	ASTM D 4253-00	18.47
Minimum dry unit weight (kN/m ³)	ASTM D 4254-93	14.4
Maximum void ratio	ASTM D 4254-93	0.73
Minimum void ratio	ASTM D 4253-00	0.485
Soil Classification according to (USCS)	ASTM D 422-07	SW
Friction angle	ASTM D 3080-03	Loose state: $\phi = 33^\circ$
Friction angle	ASTM D 3080-03	Medium state: $\phi = 36^\circ$
Friction angle	ASTM D 3080-3	Dense state: $\phi = 39^\circ$

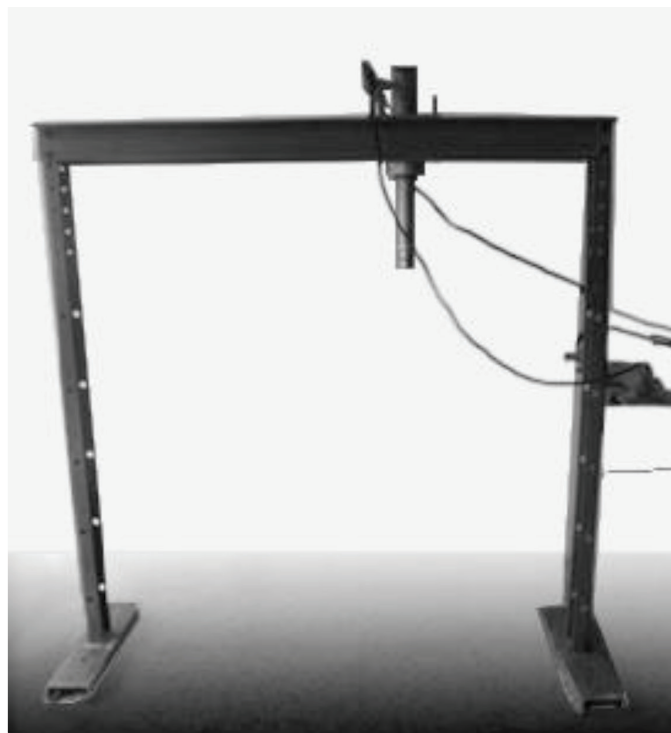


Fig. 1 Steel loading frame and hydraulic jack system.

height of 1.8 m. The maximum load that could be applied was about 5 tons. A manual system was fixed on the right column of the frame to control the hydraulic intensity. A plastic tube was used to pump the hydraulic oil from the manual system to the piston. An abreaction system was used to expel the air from the hydraulic system.

2.2 Soil preparation and measurement devices

The method used to deposit the soil in the testing tank at a known and uniform density was based on a method developed by Koulbuzewski (1948), which is known as the “raining technique”. A moveable steel tank 300 mm long, 300 mm wide, and 450 mm high and ending in an inclined funnel system out-



let, was mounted above the testing tank, and used as a hopper to pour the testing material from different heights. A simple sliding system of a perforated plate was provided in the outlet of the funnel to start or stop the raining of the soil.

The “raining technique” applied to deposit the soil in the testing tank at a known and uniform density was used in preparing the test soil. An inclined funnel was mounted above the testing tank and used as a hopper to pour the testing material from different heights through two rollers. In order to facilitate the horizontal movement of the steel tank, a simple sliding system was prepared for this purpose.

The height of the drop and the rate of discharge of the sand mainly affect the density of the sand layer in the raining method (Turner and Kulhawy, 1987). Two rollers fixed at the top of the box were used to adjust the height of the raining device



a. Pressure cells.



b. Pressure cell placed on the pipe surface.

Fig. 2 Total earth pressure cells.

to control the height of the free fall of the sand. Several trials with different heights of the falls were performed in order to achieve the relative density desired.

The depth of the embedment of the pipe was adjusted by limiting the top of the tank by means of a temporary wooden wall. The free end of the pipe was then sealed, and the soil was poured into the tank. The surface of the soil was then leveled. At this stage, the loading plate (a steel plate with dimensions of 800 x 200 x 20) mm was placed on the surface of the soil at the desired position, and the system was ready for testing under different loading conditions.

A data acquisition system was used by which all the strains could be read and automatically recorded. The system is able to read the data from eight channels simultaneously. Three contact pressure transducers were used to scan the normal and shear stresses on the base and sides of the pipe. A displacement transducer was used to detect the surface settlement. A load cell was also placed in the loading shaft to detect the pattern of the loads applied on the footing surface accurately.

A number of pressure cells, 100 mm in diameter, were used to measure the total pressure acting on their point of installation. Figure 2a shows the earth pressure cells used in this study. Figure 2b illustrates the method of the installation of the pressure cell above the pipe. The pressure cell consists of two circular stainless-steel plates welded together around their periphery, leaving a narrow cavity between them. The cavity is filled with antifreeze fluid. Model CF-350-20AA-(II)-C20 strain gauges were used in all the tests. A gauge with a length of 20 mm was designed for measuring the strain on the surface of the PVC pipe.

The geogrid material used in this study is Pars Mesh Polymer (PMP), Type SQ12. Tension tests were performed on the geogrid as per ASTM D6637 to determine its strength and tensile modulus. The tensile modulus (the secant slope of a stress-strain curve) was found to be 79 kN/m², and its yield strength was taken as 0.47 kN/m at a 10% strain level. The aperture size was 14x20 mm.

Testing Procedure

The preparation of the soil bed (bedding materials) requires that any stable soil will act adequately as a bedding material, provided that it is placed and compacted around a pipeline. From a practical point of view, granular material is more read-

ily compacted; it has become widely accepted. The bedding material should be of a similar particle size to that in the trench sides. In the present study, a layer of sand with a 300 mm height was well compacted before the laying of the pipe, which was instrumented by four strain gauges in four places on the perimeter of the pipe opposite and connected to a strain indicator by wire. Then one pressure cell was placed horizontally on the crown of the pipe. The second cell pressure was put vertically on the side of the pipe, while the last one was placed horizontally beneath the surface of the sand by the same width of strip footing in case there was no geocell reinforcement or geogrid. When they exist, a third soil pressure cell will be placed beneath them. Figure 3 shows the schematic diagram of the buried pipe system model with its instrumentation.

After the instrumented pipe was laid, the sand was then poured by hopper according to the height to achieve the required density off either dense, medium or loose, and with three embedment depths (1.5, 2, 3) times the diameter of the pipe. When the sand level reached the required height, then the strip footing, which was made from steel plate with a thickness of 20 mm and a width and height off 110 x 750 mm, was placed. Throughout the test, the soil surface settlement was monitored by two LVDTs placed on the opposite edges of the loading plate to measure the average settlement of the surface. The pipe wall's circumferential strain was recorded by four strain gauges on the external crown and sides of the pipe invert. Then, a load cell was placed at the center of the strip loading plate under the piston of the hydraulic jack. Weirs connected with channels were checked before the indicator started to apply the loading with constant increments of 1 kPa. All the strain readings were transferred automatically to a laptop; the other readings were manually recorded for the three soil pressures and two LVDTs.

Three soil pressure cells were placed around the pipe; one of them was placed above the pipe in the horizontal direction, the second was placed next to the pipe in the vertical direction, while the third one was put beneath the loading plate at a depth equal to the width of the loading plate as shown in Figure 3. The loading plate was placed on the surface of the model. The test was first carried out without a geogrid. Then the test was repeated with the geogrid. Once its width was equal to 1B, then it was equal to 2B, where B is the width of the loading plate (footing). The soil pressure cell was placed directly beneath the geogrid.

3 Results and Discussion

3.1 Effect of the soil density on the pipe

Figure 4 shows the effect of soil density on the crown strain at an embedment depth equal to 2d, where d is the diameter of the pipe. It can be seen from this figure that the maximum deformation occurs on the crown of the pipe. Referring to Figure 4, when the burial depth equals 2d, the maximum strain on the pipe crown increases by about 26.6 % when the sand changes from dense ($D_r = 75\%$) to medium ($D_r = 60\%$) and by 35.3 % when it changes from dense to loose ($D_r = 35\%$). These results are expected and compatible with those obtained by Hosseini and Moghaddas Tafreshi (2002), who conducted laboratory tests of small diameter pipes buried in reinforced sand and de-

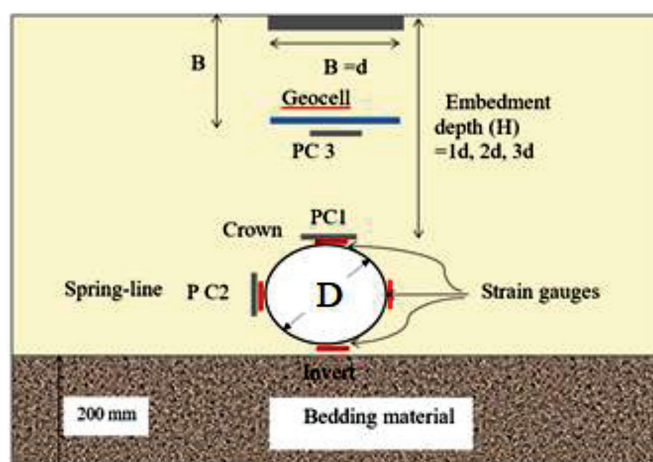


Fig. 3 Scheme of the model.

terminated that the deflection behavior and failure mechanism of the system highly depend on the soil density.

The results are also compatible with the findings of Arockiasamy et al. (2006) who studied the soil-pipe interaction and noted that whatever the soil is well compacted, it absorbs the bulk of the load transferred to the pipe and thus reduces the strain of the pipe's wall. However, in relatively loose soils, due to the weak contacts and poor interlocking of the grains and the special arrangement of the soil fabric, regardless of the embedment depth, even under low loads, the failure of the system usually occurred in a low load applied due to local buckling or the large deflection of the pipe together with excessive settlement of the loading plate.

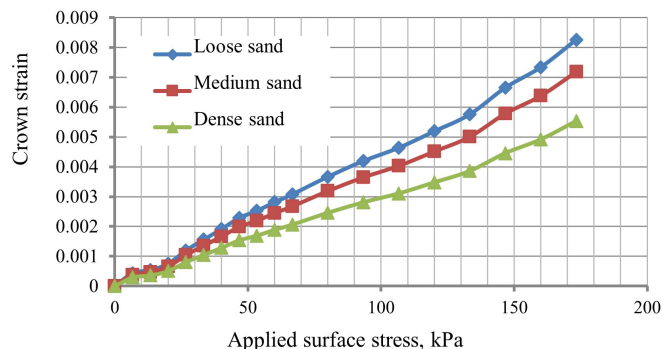


Fig. 4 Effect of the soil density on the crown strain of a pipe embedded at a depth of $2D$.

Figure 5 shows the effect of the buried soil density on the pipe invert strain. It can be seen that the invert strain is less than the crown strain by about 75%. This is because the movement of the bottom of the buried pipe is restricted by the soil, where the lateral movement is limited by the soil, which the pipe tries to push, and the axial movement is limited by the soil-pipe friction. The decrease in the strain with the increase in the soil density can also be clearly seen. The maximum strain on the pipe invert decreases by about 4 % when the sand changes from loose to medium and by 23.5 % when it changes from loose to dense when the embedment depth equals $2D$. This explains the decrease in the effect of the density on the strain as we move away from the loading source. All these percentages were calculated for the last load increment.

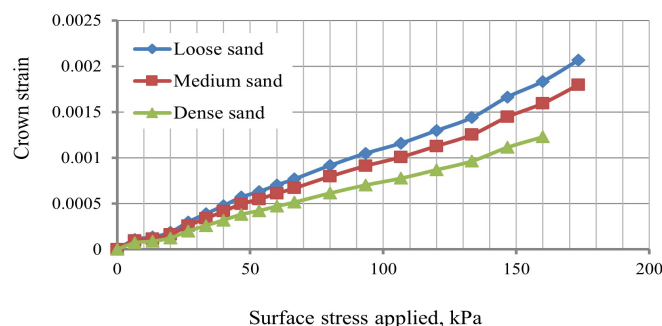


Fig. 5 Effect of the soil density on the invert strain of a pipe embedded at a depth of $2D$.

A good understanding of the behavior of buried UPVC pipes is obtained from measurements of the pipe wall strain, which indicates variations in the deformed shape of the pipe. The springline area is the edge side of the pipe in the diameter area, and there are two sides for the pipe. So in this section,

when the term “springline strain” is used, it means the average value of the right and left spring line strains. Figure 6 displays the effect of the soil density on the springline strain at different burial depths; it is evident from the results that increasing the backfill quality affords the pipe greater protection in terms of circumferential strain. The springline strain decreases by about 7% when the sand changes from loose to medium and by 22% when it changes from loose to dense for an embedment depth equals to $2D$. It can also be seen that the sign of the springline strain is negative or, in other words, it is the opposite sign of the crown strain, which explains that the pipe is suffering from compression in the crown while it is exposed to tension in the springline area.

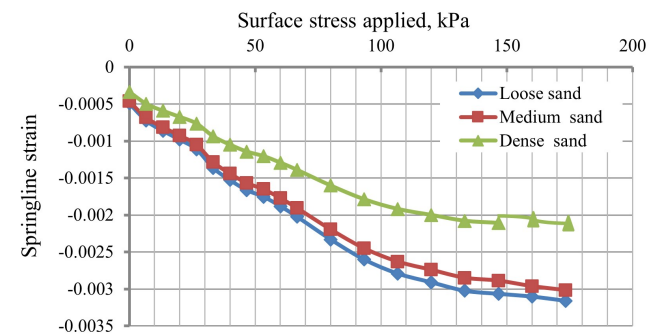


Fig. 6 Effect of the soil density on the springline strain in a pipe buried at a depth of $2D$

Figures 7 to 9 present the variation of the crown strain with a surface load applied for the pipe buried in loose, medium, and dense sand, respectively. The pipe was embedded at a depth of $1.5D$, which is kept constant to investigate the effect of the geogrid width placed at a depth of $0.5B$ below the loading plate. The geogrid location is also kept constant. The results show that the reduction in the crown strain in loose sand, when the geogrid width equals $1B$, is 13% compared with the test results without a geogrid, but this percentage decreases to 11 % when the geogrid width equals $2B$. While the reduction percent in medium sand becomes 14 % and 12% when the geogrid width was B and $2B$, respectively, in dense sand, it becomes 15% and 11% when the width of the geogrid changes from B to $2B$. These results are attributed to the full interaction that will ripen between the soil beneath a geogrid and the geogrid itself so that the pressure transferred through the system is small.

When the relative density increases, the percentage of the reduction increases because the friction and interaction between the soil and geogrid has increased; therefore, the deformation was lower. But when the geogrid width increased, the pipe showed a somewhat lower reduction in the strain. This behavior was also noticed by Mehrjardi et al. (2012a), who observed that the geocell layer was pulled down under the plate (footing) settlement; however, at a remote distance from the loading plate periphery, no tension in the geocell was observed. Therefore, the reinforcement efficiency in reducing the deformation of the pipe lessened as the width of the geogrid increased. This is also compatible with the findings of Marto et al. (2013), who studied the effect of the geogrid reinforcement on the bearing capacity properties of soil under a static load and found that the presence of a geogrid in the soil makes the

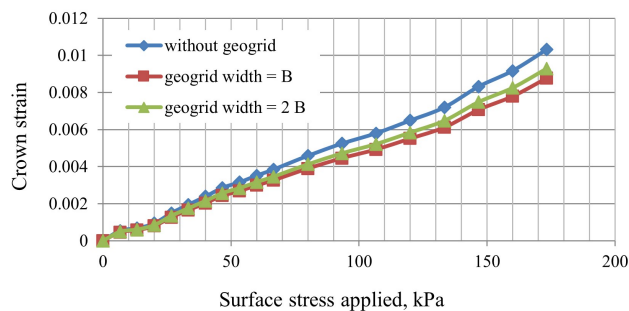


Fig. 7 Effect of the width of the geogrid on the crown strain of a pipe embedded in loose sand.

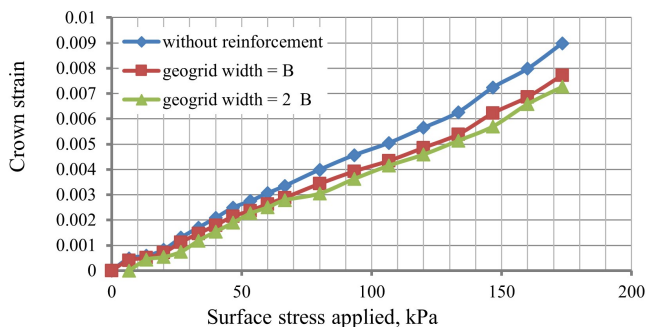


Fig. 8 Effect of the width of the geogrid on the crown strain of a pipe embedded in medium sand.

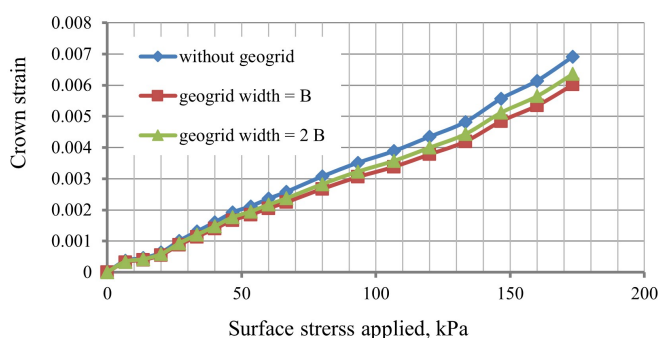


Fig. 9 Effect of the width of the geogrid on the crown strain of a pipe embedded in dense sand.

relationship between the settlement to the pressure applied of the reinforcement soil almost linear, and they further noticed that the efficiency of the reinforcement in reducing the deformation was lowered as the width of the geogrid increased.

3.2 Effect of a reinforcement on stresses in soil around a pipe

In general, soils possess a low tensile strength. The main objective of strengthening the soil mass is to increase the bearing capacity, improve the stability and decrease settlements and lateral deformations. One of the approaches is the use of polymeric materials. Geosynthetic is a well-known technique in soil reinforcement. Its use can significantly improve the soil's performance and reduce costs in comparison with conventional designs. In the present study, two types of reinforcement were used: a geogrid and a geocell. A geogrid is made from polymer materials; it may be woven or knitted from yarns, heat-welded from strips of material, or produced by punching a regular pattern of holes in sheets of material and

then stretched into a grid (Khatib, 2010). A geocell is a honeycomb with a three-dimensional cell structure that provides for the containment of compacted fill soils. The reinforcement decreases the lateral movement of the soil particles and forms a mat or rigid layer for the distribution of loads applied to a wider area footing (Dash et al., 2003).

Figures 10 to 12 show the relation between the surface pressure applied with the soil pressure measured by a soil pressure cell when the pipe is buried in loose sand, while Figures 13 to 15 and Figures 16 to 18 show the same relation when the pipe is buried in medium and dense sand, respectively.

In all the tests, the reinforcement depth was kept constant and equal to B . The improvement of the soil strength when a geogrid and geocell are used in the soil is clear.

It can be seen from Figure 10 that the vertical stress above the pipe buried in loose sand, is reduced by 8.3% when the geogrid width is $2B$, while it is reduced to 22% when the geogrid width is B . When the geocell was used, the percent reduction is equal to 62%.

On the other hand, Figure 13 shows the percent reduction in the vertical stress above the pipe buried in medium sand; the reduction is 28% when the geogrid width was $2B$ and 40% when the geogrid width was B and equal to 68% when a geocell was used.

Figure 16 shows that the reduction in the vertical stress above the pipe buried in dense sand is 32.7% when the geogrid width was $2B$, 42.3% when the geogrid width was B , and 75% when the geocell was used.

It can be concluded from these percentages that the efficiency of a geocell in reducing the stress transfer to the buried pipe is more than the efficiency of a geogrid; this is because the structure of a geocell and its third dimension, which gives the soil more interaction with the geocell and prevents it from sliding by confining the soil between the wall of the geocell and the soil, which leads to an increase in the strength and thus reduces the transmission of stress to the pipe.

The geocell reinforcement keeps the encapsulated soil from being displaced from directly beneath the load applied by confining the material by hoop action in the cell walls, thereby increasing the shear strength of the composite system. The load redistribution that occurs within the confined zone involves a three-dimensional interaction between the sand infill material and the cellular structure as stated by Moghaddas Tafreshi and Dawson, (2010).

Vertical stress applied to the infill induces horizontal active pressure at the perimeter of the cell. The interface friction of the infill wall transfers the load into the cell structure which, in turn, mobilizes the resistance in the surrounding the cells. It is also evident that the cells that surround a loaded cell offer greater passive resistance due to the greater lateral strain in the vicinity of the load. The combined effect of these mechanisms acts like a large mat that spreads the applied load over an extended area, instead of directly at the point of contact, and thus provides a composite lab with high flexural stiffness and load support capabilities within the geocell reinforcement, consequently leading to an improvement in the overall settlement performance.

Figure 11 shows the effect of the reinforcement on the horizontal pressure beside the pipe buried in loose sand. The percent reduction in horizontal stress is 48% when the sand was reinforced by a geogrid of width B , 50% when the geogrid width was $2B$, and 75% when a geocell was used. Figure 14 shows

the same relation but in medium sand, and the percent of the reduction in horizontal stress was 28.6 % when the geogrid width was B, 42.8% when the geogrid width was 2B, and 85 % when the geocell was used. Also Figure 17 shows that the percent reduction in the horizontal stress in dense sand is 66 % when the geogrid width was B, 75% when the geogrid width was 2B, and 90 % when a geocell was used instead of a geogrid.

Figure 12 presents the effect of reinforcement on the vertical stress under the reinforcement in loose sand; the percent reduction in the stress was 20 % when the geogrid width was B, 28 % when the geogrid width was 2B, and 57% when a geocell was used.

Similarly, Figure 15 shows the percent reduction in the vertical stresses beneath the reinforcement embedded in medium sand; the percent reduction when the geogrid width equals B is 24 %, 33 % when the width equals to 2 B, and 60 % when a geocell was used.

Figure 18 shows the effect of reinforcement on the vertical stress under a reinforcement in dense sand; the percent reduction in stress was 27 % when the geogrid width was B, 37% when the geogrid width was 2B, and 67 % when the geocell was used.

It can be seen that when the density of sand increased, the percent of reduction increased, which can be interpreted by the interlocking of the sand particles with the reinforcement, which gives a more strength to bear the loads applied.

Obviously, the use of a geocell as a reinforcement in loose sand can reduce the vertical pressure on the pipe crown. The pattern of the strain variations in the geocell walls indicated that the geocell mattress behaves as a composite beam supported by the subgrade soil. The load dispersion in the geocell mattress was found to be governed by factors such as geometry of the geocell layer and the position of its placement under the footing.

When the geocell reinforcement is filled with soil, it appears to behave as a stiff bed, which redistributes stress over a wider area, instead of a narrow stressed area at the point of the load applied when no reinforcement is used.

The geocell reinforcement keeps the encapsulated soil from being displaced from directly beneath the load applied by confining the material by hoop action in the cell walls, thereby increasing the shear strength of the composite system. The load redistribution that occurs within the confined zone involves a three-dimensional interaction between the infill materials and the cellular structure. The vertical stress applied to the infill induces a horizontal active pressure at the perimeter of the cell. The interface friction of the infill wall transfers the load into the cell structure which, in turn, mobilizes resistance in surrounding cells as indicated by Mehrjardi et al. (2012b).

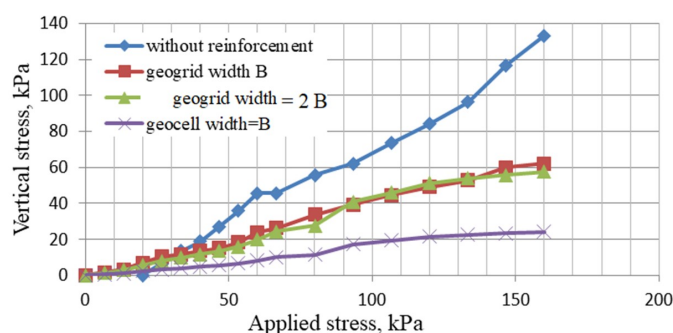


Fig. 10 Effect of a geogrid on the vertical stress above a pipe embedded in loose sand.

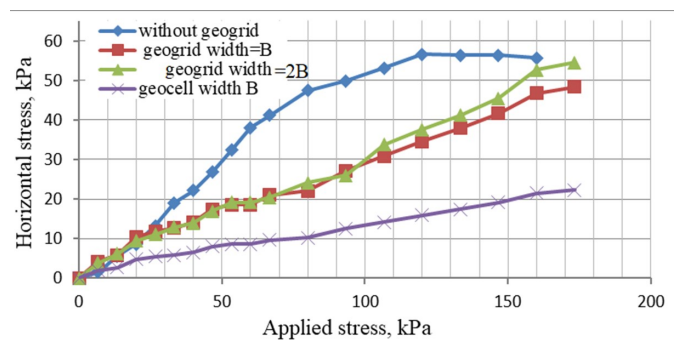


Fig. 11 Effect of a geogrid on the horizontal stress beside a pipe buried in loose sand.

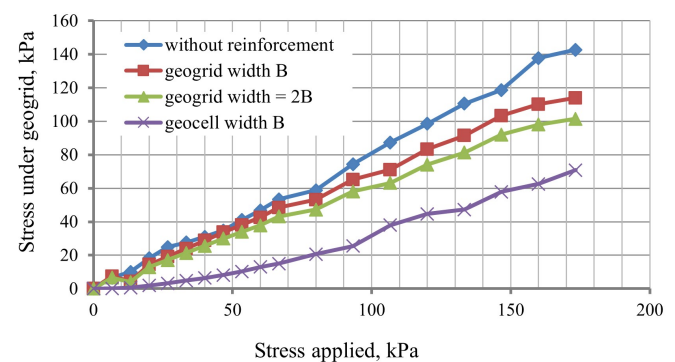


Fig. 12 Effect of reinforcement on the vertical stress under a geogrid embedded in loose sand.

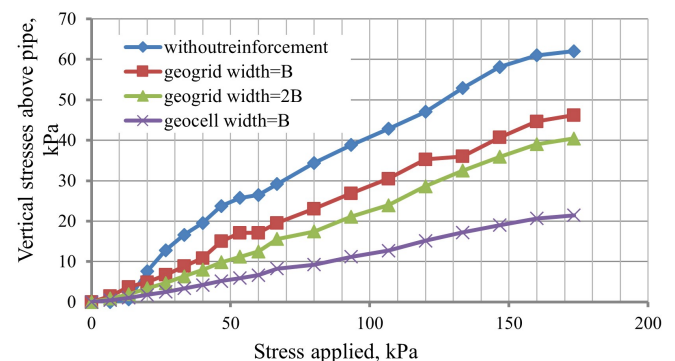


Fig. 13 Effect of reinforcement on the vertical stresses above a pipe embedded in medium sand.

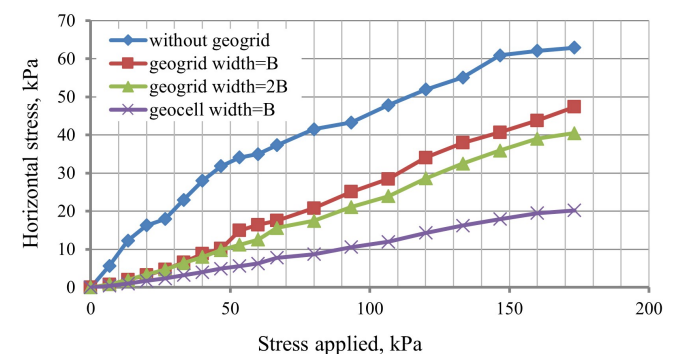


Fig. 14 Effect of reinforcement on the horizontal stress beside a pipe embedded in medium sand.

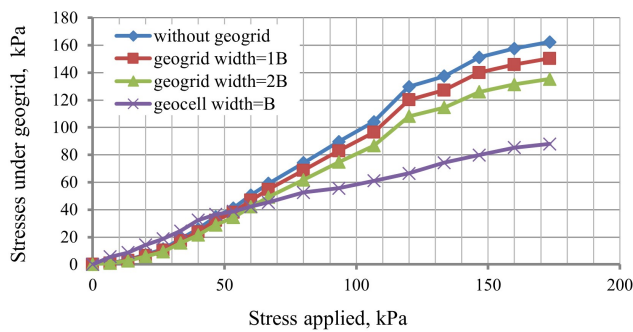


Fig. 15 Effect of a geogrid on the vertical stresses under a geogrid embedded in medium sand.

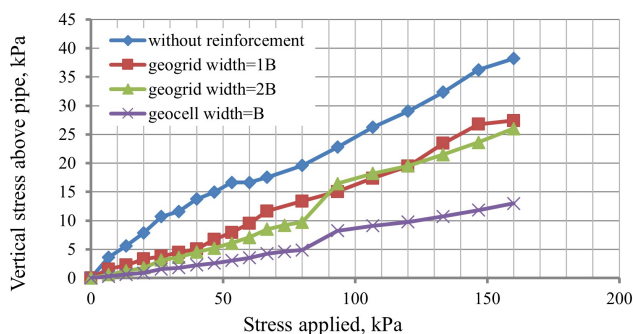


Fig. 16 Effect of reinforcement on the vertical stresses above a pipe embedded in dense sand.

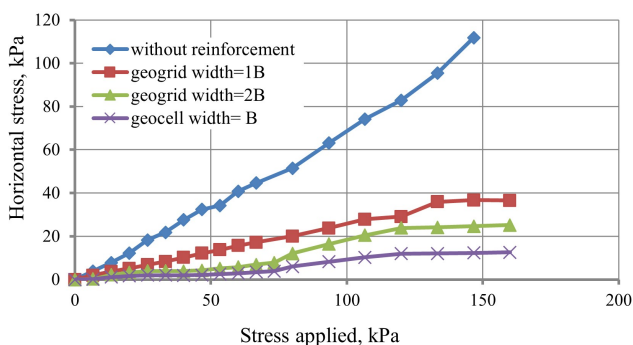


Fig. 17 Effect of reinforcement on the horizontal stresses beside a pipe embedded in dense sand.

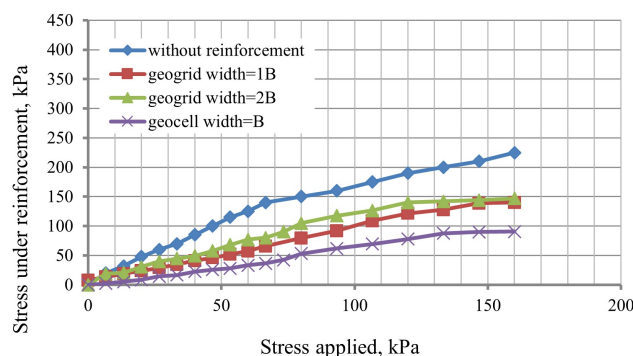


Fig. 18 Effect of a geogrid on the vertical stresses under a geogrid embedded in dense sand.

Fattah et al. (2018b) found that the vertical pressure on the crown of a buried pipe decreased between 13-41% due to reinforcement of the sub-base layer by the geocell under different load frequencies. The vertical pressure reaching the pipe was increased when the frequency was increased, for the same load amplitude, with or without a geocell.

Moghaddas Tafreshi et al. (2020) concluded that with the increase in the geocell's width beyond the optimal width of (expanded polystyrene) EPS (i.e., 1.5 times the pipe diameter), not only is no further improvement generated, but it also counteracts the beneficial effect of an EPS block, as the negative influence of the pipe's behavior would be expected with the increase in the width of an EPS block more than 2.5 times the pipe diameter. This could be attributed to diminishing the arching effect over the pipe due to the use of a wider EPS width, which extends the soil prism over the pipe (Fattah et al., 2016; Witthoeft and Kim, 2016).

3.3 Effect of reinforcement on the surface settlement

The improvement in reducing the surface settlement by adding a geogrid reinforcement is clearly seen in Figures 19 to 21, which represent the relation between the surface load applied with the surface settlement for a soil-pipe system buried in loose, medium or dense sand, respectively. It can be seen in Figure 19 that the surface settlement of the footing above the buried pipe embedded in loose sand decreases by about 25 to 35% when the geogrid width equals B , and 40 to 45% when the geogrid width equals $2B$, while this percent becomes 50 to 57% when a geocell is used.

It can be seen that the percent decrease in a settlement is increased when the geogrid width is increased, but in the small amount of 10%. This is compatible with the results reached by Marto et al. (2013), who found that the reinforcing efficiency in reducing the maximum footing settlement is reduced as the width of a geogrid was increased. They also found that if the width of a geogrid is greater than five times the width of a footing, it had no significant effect.

These results are compatible with those of Fattah et al. (2018a), who found that a geocell reinforcement greatly reduces the soil surface settlement above buried pipes. The surface settlement increases with the increase in the load amplitude. The reduction of a surface settlement due to the geocell reinforcement is about 29 to 43 % when the amplitude of load is 0.5 ton. This value became 32 to 41% when the amplitude of the load was up to 1 ton.

The geocell reinforcement keeps the encapsulated soil from being displaced directly beneath the load applied by confining the material via hoop action in the cell walls, thereby increasing the shear strength of the composite system. The load redistribution that occurs within the confined zone involves a three-dimensional interaction between the infill sand and the cellular structure.

Using a reinforcement in soil would control and decrease the vertical settlement and progressive densification would be induced. The reinforcement can contribute to increase the bearing capacity through significantly changing the geometry of the collapse pattern, thereby preventing the mechanism from reaching deep into the soil. The reinforcement prevents the most adverse mechanisms from occurring, thereby leading

to an increase in the limit load. The main role of the inclusion is to reduce the distortion rate in the sheared zone and reduce the ultimate shear stress mobilized in the shear zone. The reinforcement provides an effective restraint and has a useful role in preventing vertical spreading of the soil. As a result, the shear strength is distinctly increased and the failure pattern is modified as stated by Michalowski and Shi, (2003).

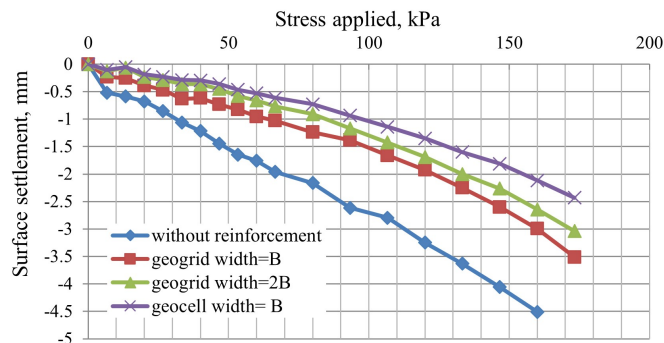


Fig. 19 Effect of a geogrid on the surface settlement above a pipe embedded in loose sand.

Figure 20 presents the surface settlement above a pipe embedded in medium sand. It can be seen that the surface settlement decreases by about 41% when a geogrid with a width of B was used, decreases by 46% when the geogrid width equals 2B, and by 60% when geocell was used. Figure 21 presents the same relation but in dense sand, which indicates that the surface settlement decreases by 45 % when the geogrid width equals B, 68% when the geogrid width equals 2B, and 79% when a geocell is used.

It can be seen that the geocell reinforcement is more efficient in reducing the surface settlement than a geogrid reinforcement, which is due to the inherent strength of the geocell material and its own structure (the third dimension of a cell), which prevents the lateral movement of the soil particles and forms a rigid slab for the distribution of loads applied to a wider area vertically. Confinement of the soil between the cells also causes an increase in the strength.

Fattah and Mohammed Redha (2016, 2022) found that reinforcing the sandy soils with a geocell leads to a beneficial reduction in the dynamic response (surface settlement, displacement amplitude, and dynamic pressure transmitted) for all the soil states at different percentages. This is accompanied by an increase in the soil strength, as well as reducing the hazards of a traffic load by decreasing the stresses transmitted to the pipe. This behavior can be attributed to the geocell reinforcement that is acting as an interconnected cage; its vertical walls are serving as a series of plate anchors, which mobilizes substantial resistance against the footing settlement, as well as increasing in the improvement performance.

4 CONCLUSIONS

- The maximum deformation occurs at the crown of the pipe. The deformation increases when the density decreases. For a burial depth equal to 1.5 d, the maximum strain on the pipe crown increases by about 22.5 % when the sand changes from dense to medium and

36.8 % when it changes from dense to loose. When the burial depth equals 2D, the maximum strain on the pipe crown increases by about 26.6 % when the sand changes from dense to medium and 35.3 % when it changes from dense to loose. The maximum strain on the pipe crown increases by about 19.35 % when the sand changes from dense to medium and 33.3 % when it changes from dense to loose.

- The invert strain in the tunnel is less than the crown strain by about 75%. The maximum strain on the pipe invert decreases by about 36 % when the sand changes from loose to medium and 39.6 % when it changes from loose to dense for the case of n embedment depth equal to 1.5. This percent becomes 4% when the sand changes from loose to medium and 23.5% when the sand changes from loose to dense for an embedment depth equal to 2D.
- The vertical stress above a pipe buried in loose sand is reduced by 8.3% when the geogrid width is 2B, while it is equal to 22% when the geogrid width was B. When a geocell was used, the percent reduction was equal to 62%. The percent reduction in the vertical stress above a pipe buried in medium sand is about 28% when the geogrid width was 2B, 40% when geogrid width was B and equal to 68%, when the geocell was used. The reduction in vertical stress above the pipe buried in dense sand was 32.7% when the geogrid width was 2B, 42.3% when the geogrid width was B, and 75% when a geocell was used.
- There is a decrease in stress beneath the geogrid layer in loose sand; the percent reduction in the stress was 20

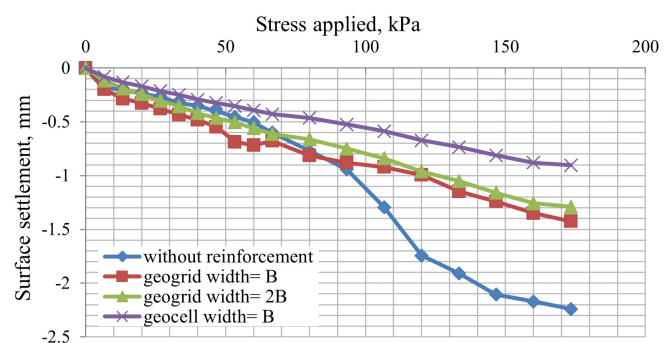


Fig. 20 Effect of a geogrid on the surface settlement above a pipe embedded in medium sand.

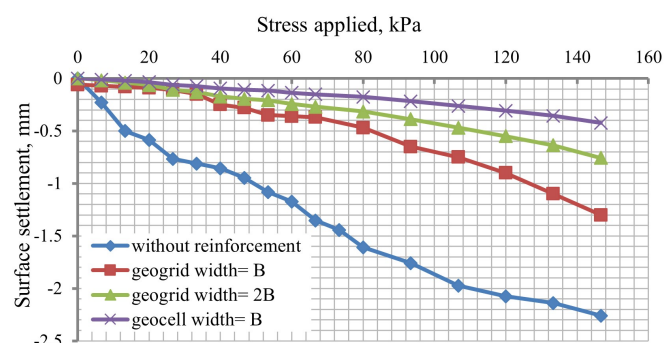


Fig. 21 Effect of a geogrid on the surface settlement above a pipe embedded in dense sand.

% when the geogrid width was B , 28 % when the geogrid width was $2B$, and 57% when a geocell was used. Similarly, the percent reduction in the vertical stresses beneath a reinforcement embedded in medium sand, the percent reduction when the geogrid width equal B was 24%, 33 % when the width equal to $2B$, and 60 % when a geocell was used. In dense sand, the percent reduction in stress was 27% when the geogrid width was B , 37% when the geogrid width was $2B$, and 67% when a geocell was used.

- The surface settlement of a buried pipe embedded in loose sand decreases by 30% when the geogrid width

equals B and 40% when the geogrid width equals $2B$, while this percent becomes 50% when a geocell is used. The percent decrease in settlement is increased when the geogrid width is increased but in a small amount (10%). The surface settlement decreases by about 41% when a geogrid is used with a width of B and decreases by 46% when the geogrid width equals $2B$ and by 60% when a geocell is used, but in dense sand the surface settlement decreases by 45 % when the geogrid width equals B , 68% when the geogrid width equals $2B$, and 79% when a geocell is used.

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HYDRAULIC CALCULATIONS OF A TELESCOPIC WATER INTAKE

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Abstract

Telescopic water intake structures allow for the selective intake of water from the top layers of a reservoir. A telescopic water intake has a high degree of mobility, and a wide field of application; at the same time, it is very simple and convenient to operate. Despite these advantages, an exact calculation method for a telescopic water intake is lacking (Lipin, 2020; Lipin, 2022). Therefore, the purpose of this research is to develop an exact calculation method for a telescopic water intake. In order to perform this task, all the structural elements of a telescopic water intake were analyzed separately by means of existing hydraulic principles. The impact of the critical submergence depth of the water intake funnel on the operability of a telescopic water intake was studied, and an equation for the critical submergence depth calculation was proposed. The optimal range of the flow rate coefficient ($\mu = 0.30 - 0.45$) was proposed for calculating the funnel flow rate, and the correlation between the flow rate and funnel inlet diameter was defined. The optimal curvature radius of the elbow was proposed. In order to verify the above mentioned parameters and correlations, analytical and numerical calculations of specific examples were performed. The results of the numerical modelling demonstrated that the critical submergence depth and shape of the water intake funnel, as defined by the analytical calculations, ensured its effective operation free of cavitation and vortices. The analytically calculated dimensions of the telescopic column, following the numerical modelling results, provided normal hydraulic conditions without active cavitation. According to the numerical modelling, the proposed curvature radius of the elbow was sufficient to avoid significant cavitation and vortex formation in the elbow structure. This research can provide guidance for the design of telescopic water intake structures.

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Key words

- Water,
- Structure,
- Calculation,
- Funnel,
- Elbow,
- Fluent,
- Finite element method.

1 INTRODUCTION

The main purpose of the telescopic water intake is intake of water from top layers of the water source. Top layers of water are free of sediments and have higher temperature because of

direct heat of sun rays, evaporation and etc. Telescopic water intake works in automatic regime according to the depth changes in the water source. It means, the head above the crest of the funnel is constant and does not change with depth fluctuations.

Traditionally, specialists involved in the design of surface water intakes have faced the problem of vortex formation. Therefore, in order to understand the physical processes occurring in a vertical telescopic water intake funnel, we consider it appropriate to review the processes of vortex formation.

Most scientists who have studied the formation of vortices noted the importance of the Froude number of the water intake for the intensity of the formation of vortex processes. However, in addition to the Froude number, the dimensions of the structure, especially the pressure pipeline, the depth of the water intake, the volumetric circulation of the suitable flow and the ratio of the head on the crest of the funnel to its diameter (H/D) are also important (Jain et al, 1978). Although the above parameters are important for all types of water intakes, they are especially important for submerged water intakes, where an insufficient depth of penetration can lead to the formation of vortex processes and where flow rate losses can be significant compared to other types of water intakes.

The term „critical submergence depth“ (H_c) was introduced in 1978 by Jain (Jain et al, 1978) as the minimum depth at which strong and unwanted eddies do not form. If the submergence depth is less than H_c , a vortex will form, but this is not always true in the opposite case, since insignificant vortices can form with an increasing depth.

Studies have established the relationship between the submergence depth and vortex formation to provide guidance for the design of surface water intakes. They mainly considered vortex flows that entrain air, since these processes are the main cause of the destruction of hydraulic structures. In 1987, Hecker (Hecker, 1987) proved that the air is most of all en-

trained by the 5 and 6 types of vortex flow (Fig. 1).

One of the earliest recommendations for the design of pumping stations was done by (Prosser, 1977). It was said that the flow should be uniform across the entire width of the channel at a speed of no more than 1 m/s; the supports should be streamlined to avoid flow separation; stagnant areas of the flow should be filled; and debris traps should work as flow straighteners. He also stated that the factors below are the result of poor design and should be avoided: abrupt changes in flow direction, steep slopes, and non-streamlined supports. If all the conditions proposed by Prosser are met, then the minimum submergence depth of the water intake is determined by the equation:

$$H = 1.5 D, \quad (1)$$

where D is the diameter of the inlet part of the funnel (m); H is the funnel crest head (m).

(Reddy and Pickford, 1972) were the first who used the relative head to study the probability of the formation of vortex processes. They determined that the main free surface eddies occur when the relative head is less than:

$$H/D = 1 + Fr, \quad (2)$$

where Fr – the Froude number.

Yildirim and Kochabas (1995) conducted a study of the critical depth in the water intake during the formation of a vortex with an air barrel. To form a flow zone, theoretically, a funnel was combined with a uniform channel flow. In this study,

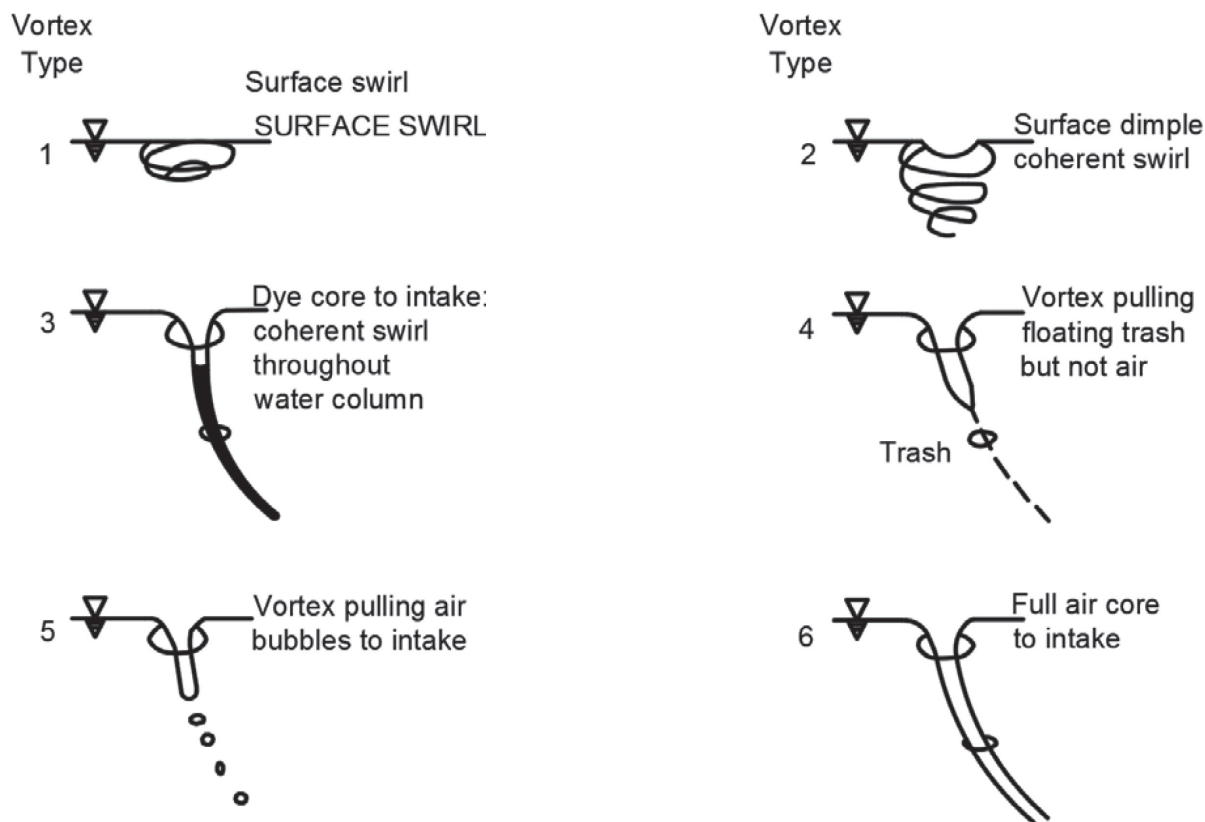


Fig. 1 Classification of types of vortex funnels (Hecker, 1987)

the funnel flow was equated to the uniform flow to ensure continuity of the system. In experiments, the critical depth was equated to the funnel diameter (Yildirim and Kocabas, 1995).

$$\frac{H_c}{D} = \frac{1}{2\sqrt{2}} \left(C_d \frac{V}{U_\infty} \right)^{1/2}, \quad (3)$$

where C_d is the vertical clearance of the point C to the canal bottom (m); V is the average intake velocity (m/s); U_∞ is the cross-sectional average velocity of the incoming flow (m/s).

Gordon 1970 identified four main criteria that determine the filling properties of a vortex in an intake: the geometry of the approach flow to the intake, the velocity in the flow region, the critical depth, and the size of the intake (Gordon, 1970). Twenty nine water intake structures were studied, and a formula for industrial use was obtained. Taking into account the hydraulic features of the flow, two equations were developed; for the symmetrical case, the equations have the following form:

$$\frac{H_c}{D} = 1.7Fr, \quad (4)$$

Experimentally, Vadoud derived an equation for determining the critical submergence depth of a simple vertical intake (Vadoud et al, 2013):

$$H_c = 3Fr^{0.248}, \quad (5)$$

This expression is valid for Froude numbers in the range of 0.68 – 2.86.

MATERIALS AND METHODS

Analytical method for calculating a telescopic water intake

The hydraulic calculation of a vertical telescopic water intake consists of the following elements: funnels; a telescopic column; float; elbow; outflow tract; stilling well (Gasarov and Lipin 2021) (Fig. 2). A stilling well is required for dissipation of the flow energy downstream when transporting in an open way, i.e., using the channel. The calculation of the outflow tract and stilling well is not considered in this work.

The calculation of the water intake is carried out in the following sequence: firstly, we calculate the funnel. The water intake funnel is one of the main structural elements of a vertical telescopic water intake. The efficiency, durability, and profitability of the entire intake structure depend on the correct design of the funnel.

In a funnel-shaped water intake with a decrease in the cross-section, the flow velocities increase, and the pressure along the central axis of the water intake decreases. In this case, if the pressure along the central axis of the water intake does not fall below atmospheric, then vortices are not formed. Thus, the phenomenon of vortex processes is formed from the interaction of the water intake's shape, the depth of the water intake, and the properties of the liquid, such as viscosity and the surface tension.

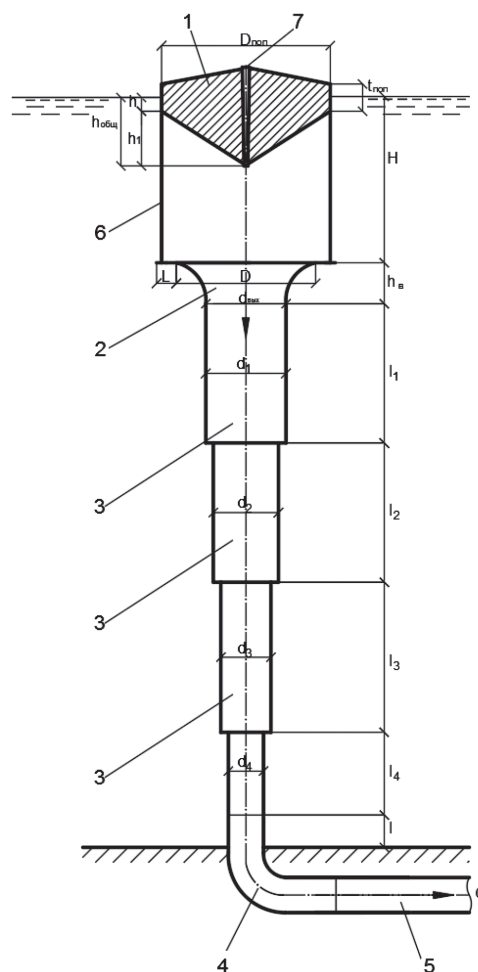


Fig. 2 Scheme for calculating the telescopic water intake: 1-float, 2- inlet funnel, 3- telescopically connected pipes (tapering downwards), 4- elbow, 5- discharge pipe, 6- connecting elements, 7- conical air supply tube

Numerous research works were analyzed in order to determine critical submergence depths and vortex formations in a funnel. As a result of these studies, different formulas for calculating the critical submergence depth were developed (Prosser, 1977; Reddy and Pickford 1972; Yildirim and Kocabas 1995; Wu et al., 2006; Vadoud et al, 2013). In our research we use Prosser's equation (1) in order to study hydraulic processes inside a funnel (Prosser, 1977).

The main parameters for the design of a telescopic water intake funnel are the inlet diameter of the funnel (D) and the funnel crest head (H). By means of these parameters and the μ flow rate coefficient, the funnel flow rate can be defined by the equation (Shterenlikht, 2004; Uginchus and Chugaeva, 1971):

$$Q = \mu \cdot \frac{\pi \cdot D^2}{4} \cdot \sqrt{2 \cdot g \cdot H}, \quad (6)$$

where Q is the flow rate of water taken by the water intake (m^3/sec); μ -flow rate coefficient, dimensionless; H funnel crest head above, m; $g=9.81$ (m/s^2) is the acceleration of gravity.

For purposes of the calculations, the funnel should be realized as a flooded nozzle having a conical or conoidal shape. In textbooks on hydraulics for a submerged conical nozzle, the flow rate coefficient should be taken as $\mu=0.85-0.87$ at an angle at the top of the funnel cone $\Theta=30-50^\circ$, for a conoidal

nozzle at large Reynolds numbers $\mu = 0.97-0.98$ (Shterenlikht, 2004). In practice, conoidal nozzles do not occur due to their complicated shape and difficulty of manufacturing; in most cases, they are replaced by a close-shaped nozzles assembled from several truncated cones.

It should be noted that a tower spillway with a flooded funnel has a similar operating principle as a telescopic water intake. Only a handful of works have been devoted to the study of spillways, most notably, those by Mois (Mois, 1959; Mois, 1970). Mois noted a dependence between the flow rate coefficient and relative head (Mois, 1970), i.e., as the flow rate coefficient increases, the ratio between the inlet and outlet diameters of the funnel decreases.

In practice, there is no calculation of a funnel flow rate with a float located at the top. We carried out analytical calculations and verified the results by means of numerical modelling in order to investigate the hydraulic processes occurring in the funnel of the telescopic water intake. According to Mois, the ratio of the inlet and outlet diameters of the funnel decreases as the coefficient of the flow rate increases. Based on the results of the analytical calculations and the numerical modelling of a large number of models with different values of the flow rate coefficient, it was concluded that the particles of water entering the funnel acquire a very large horizontal component of the velocity with an increase in the value of the flow rate coefficient. This results in the separation of the flow from the funnel walls and the formation of significant cavitation processes, that interfere with the normal operation of the structure. It was found that the use of a flow rate coefficient in the range $\mu = 0.30 - 0.45$ together with height of the funnel equal to the inlet diameter $h_{fun} = D$ and submergence depth of the funnel crest equal to 1.5 inlet diameter of the funnel $H = 1.5D$ can ensure normal hydraulic conditions without vortex formation and avoid the formation of significant cavitation, hence, protecting the funnel from erosion. In this research for the analytical calculation of the funnel flow rate, we accept the flow rate coefficient $\mu = 0.36$. Thus, considering equation (1) and the constants in equation (6) we get:

$$D = 0.85 Q^{\frac{2}{5}}, \quad (7)$$

It should be noted that the water flow rate Q taken from the source is set according to the water consumption. In order to connect the funnel inlet with the float behind the funnel crest, an initial section with a length of $L = (0.4 - 0.5) D$ is arranged. In this case, taking into account the length of the initial section, the diameter of the funnel is determined by the equation:

$$D_{fun} = D + 2L, \quad (8)$$

In order to determine the diameter of the outlet part of a funnel, the height of the outlet part is assumed to be equivalent to the diameter of the inlet part, for the research purpose, i.e., $h_{fun} = D$, and the diameter of the outlet part of the funnel is thus determined by the equation:

$$d_{out} = \sqrt{\frac{4Q}{\varphi \pi \sqrt{2g(H+h_{fun})}}}, \quad (9)$$

where d_{out} is the diameter of the outlet part of the funnel (m);

φ is the velocity coefficient (dimensionless, in our case $\varphi = 0.96$); H is the funnel crest head above (m); and h_{fun} is the head from the funnel outlet level to the funnel crest (m).

The diameter of the first telescopically connected pipe is accepted to be equal to the diameter of the outlet part of the funnel, i.e., $d_1 = d_{out}$.

The diameter of the second pipe (d_2) is determined by the equation:

$$d_2 = d_1 - 2(\delta + a), \quad (10)$$

where d_1 is the inner diameter of the first pipe (m); δ is the thickness of the wall of the second pipe, (mm); a is the gap between the walls of the pipes or the thickness of the support ring (mm).

The diameter of the subsequent pipes is determined by equation (10) according to the diameter of the previous pipe and the thickness of the wall, for example, the diameter of the third pipe is determined as follows: $d_3 = d_2 - 2(\delta + a)$.

The elbow is an important structural element of the telescopic water intake; the efficiency of the entire intake structure is dependent on its reliability. Under the action of centripetal acceleration arising when the flow turns in the plane of formation of its velocity structure, centrifugal forces arise, increasing the pressure on the convex surface and decreasing the pressure on the concave surface. As a result, the pressure can drop so much, on the convex surface of the elbow, that intense cavitation processes of destruction may occur.

The diameter of the elbow and the outlet pipeline is taken to be equal to the diameter of the last pipe, i.e., $d_{elbow} = d_{last} = d_{out,pipe}$. The last pipe connected to the elbow is considered to be fixed.

To prevent cavitation processes and vortex formation in the elbow a large number of numerical models with different curvature radii were previously studied. According to the recommendations available in the literature, the curvature radius should be assigned in the range $R_0 = 2 - 5d_{elbow}$. To study and prevent cavitation, we propose the curvature radius:

$$R_0 = 2.5d_{elbow}, \quad (11)$$

By means of computer modelling, we can study the hydraulic processes occurring in the elbow.

Calculation of the float

The calculation of the float consists in determining its buoyancy, i.e. its operability, taking into account the load imposed on it, and thereby determining the depth of the immersion, height and diameter of the float.

According to Archimedes' law, to ensure the buoyancy of the float, the lifting force P_f is equal to the force G , which tends to immerse it in water:

$$P_f = G, \quad (12)$$

The lifting force (N) is determined by the formula:

$$P_f = \gamma \omega h = \gamma W, \quad (13)$$

where $\gamma = \rho g$ is the volumetric weight of the water (kg/m^3); ω is the cross-sectional area of the float, limited by the wetting

contour (m^2); h is the immersion depth of the float (m); W is the volume of water displaced by the float or the volumetric displacement (m^3); and ρ is the density of the water (kg/m^3).

The force tending to immerse the float in water (G) includes the weight of the float itself, the funnel, telescopically connected pipes (hanging pipes) and other elements of the device. The weight of each element is determined by the following formula:

$$G_i = g \Sigma \rho_i W_i = g \Sigma \rho_i S_i \delta_i ; (i = 1, 2, \dots, n), \quad (14)$$

where ρ is the density of the material from which the element is made (kg/m^3); W_i is the volume of material, m^3 ; S_i is the element's surface area (m^2); δ_i is thickness of the wall's material (m).

Considering (13) and (14), equation (12) has the form:

$$\gamma W = \Sigma G_i, \quad (15)$$

Through equation (15), the buoyancy or operability of the float is established, and its dimensions are also determined.

Displacement is determined by the shape of the float. In this specific case:

$$W = \frac{1}{3} h_1 \omega + h \omega, \quad (16)$$

where h_1 is the height of the conical part of the float, which is completely submerged in water (m); the cross-sectional area of the cylindrical part of the float (m^2); D_{float} is the float diameter (m); and h is the immersion depth of the cylindrical part of the float (m).

The displacement of the float can be calculated by equation (15) if the load acting on it is known. Considering the known value of displacement W and diameter of the float D_{float} , the immersion depth of the float's cylindrical part is determined by equation (16).

Draft i.e., the total immersion depth of the float (measured by the sum of the immersion depths of each conical h_1 and each cylindrical h in Figure 2), is calculated as follows:

$$h_{total} = h_1 + h, \quad (17)$$

It should be noted that the calculation of the float is carried out by the approximation method since the dimensions and the weight of the float are not known. Therefore, in the first approximation, the height (t_{float}) of the cylindrical part of the float is accepted, and then its weight is calculated. In case, the accepted height of the float turns out to be less or significantly higher than the calculated one during calculations, then its value should be increased or decreased and the calculations should be performed again.

The calculation of the float can be done in different ways, for instance, using the depth of the immersion to calculate its diameter. Using this method to solve the problem is more difficult.

$$\gamma \left(\frac{1}{3} h_1 + h \right) \frac{\pi D_{float}^2}{4} = \Sigma G_i, \quad (18)$$

where all symbols are the same.

Let us look at a specific example.

Example. Calculate the telescopic water intake using the following initial data. The flow rate of water into the reservoir is $Q = 7 \text{ (m}^3/\text{s)}$; the depth of the reservoir is 23.5 (m); the dead level is 6 (m); the float is cylindrical and has a conical base; the cone height is $h_1 = 0.4 \text{ (m)}$.

Solution. By using formula (7), we determine the diameter of the funnel inlet by taking the flow rate coefficient, which is 0.36

$$D = 0.85 \cdot 7^{\frac{2}{5}} = 1.85 \text{ m.}$$

By means of equation (1) we determine the head at the funnel crest:

$$H = 1.5 \cdot 1.85 = 2.78 \text{ m.}$$

Accepting the length of the initial part of the funnel $L = (0.4-0.5) \cdot 1.85 = 0.925 \text{ (m)}$ according to equation (8), we can determine the full diameter of the inlet funnel:

$$D_{fun} = 1.85 + 2 \cdot 0.925 = 3.7 \text{ m.}$$

The height of the funnel is taken to be equal to the diameter of the inlet part of the funnel $h_{fun} = D = 1.85 \text{ m}$. and, using formula (9), we can calculate the diameter of the outlet part with a velocity coefficient $\varphi = 0.96$

$$d_{out} = \sqrt{\frac{4 \cdot 7}{0.96 \cdot 3.14 \sqrt{2 \cdot 9.81 \cdot (2.78 + 1.85)}}} = 0.99 \text{ m.}$$

Considering the depth and water dead volume level in the reservoir, we accept 4 steel pipes each 5 (m) long and with a thickness of the wall $\delta = 10 \text{ mm}$.

The diameter of the first pipe is taken to be equal to the diameter of the funnel outlet part $d_1 = 990 \text{ mm}$. The diameter of the second pipe is found by the formula (10) with a gap between the walls $a = 5 \text{ mm}$ and a pipe wall thickness $\delta = 10 \text{ mm}$.

$$d_2 = 990 - 2(10 + 5) = 960 \text{ mm.}$$

By means of equation (10), we find the diameter of the third $d_3 = 930 \text{ mm}$ and the fourth $d_4 = 900 \text{ mm}$.

We accept $d_{elbow} = d_4 = d_{out, pipe}$ and, by means of equation (11), we find the elbow bending radius:

$$R_0 = 900 \cdot 2.5 = 2250 \text{ mm}$$

The float and the inlet funnel are made of steel sheets 10 mm thick. The diameter of the float is taken to be equal to the full diameter of the funnel inlet part $D_{float} = D_{fun} = 3.7 \text{ m}$.

Based on the total surface area $\omega_{total} = 28.75 \text{ m}^2$ of the float, sheet wall thickness $\delta = 0.01 \text{ m}$ and steel density $\rho = 7850 \text{ kg}/\text{m}^3$, we calculate the weight of the float using formula (14):

$$G_{float} = 9.81 \cdot 7850 \cdot 28.75 \cdot 0.01 = 22.14 \text{ kN}$$

In the same way, we find the weight of the inlet funnel $G_{fun} = 6.54 \text{ kN}$. By the diameter and wall thickness of the pipes, we find the weight of the pipes $G_{pipe} = 34.85 \text{ kN}$. Total float load:

$$\Sigma G_i = G_{float} + G_{fun} + G_{pipe} = 22.14 + 6.54 + 34.85 = 63.53 \text{ kN}$$

Using equation (13), we find the displacement of the float by considering the volumetric weight of the water $\gamma = 10 \text{ kN/M}^3$

$$W = \frac{63.53}{10} = 6.353 \text{ m}^3$$

By means of formula (16), we can determine the immersion depth of the cylindrical part of the float $D_{float} = 3.7 \text{ m}$. and $h_1 = 0.4 \text{ m}$.

$$h = \frac{4 \cdot 6.353}{3.14 \cdot 3.7^2} - \frac{1}{3} \cdot 0.4 = 0.458 \text{ m}$$

According to equation (17), we find the draft, i.e., the total immersion of the float:

$$h_{total} = 0.4 + 0.458 = 0.858 \text{ m}$$

By means of equation (18), we check the buoyancy of the float:

$$10 \left(\frac{1}{3} \cdot 0.4 + 0.458 \right) \frac{3.14 \cdot 3.7^2}{4} = 63.58 \text{ kg}$$

$$63.58 \geq 63.53.$$

Since equation (14) complies with the accepted and determined values, the calculation of the float stops.

Numerical calculations of the telescopic water intake

The hydrodynamic modeling of the telescopic water intake was done using two solvers of the ANSYS package. ANSYS FLUENT and ANSYS CFX both use the Navier-Stokes equations as the governing equations (Rutvika, 2016; Ansys Inc, 2013). The main difference between these two solvers is the way, in which they integrate the fluid flow equations and their solving strategies. In addition, ANSYS CFX uses the finite elements approach to discretize the domain, while ANSYS FLUENT uses finite volumes. ANSYS CFX uses a single method to solve the governing equations of motion (a coupled algebraic multigrid). ANSYS FLUENT offers three options to solve the governing equations, i.e., the density approach, the segregated approach,

and the coupled pressure-based method. Both solvers allow for the solving of complex hydrodynamic tasks. The purpose of using two solvers was to study the cavitation and vortex formation in various parts of the telescopic water intake by means of different solving strategies and different mesh sizes and topologies.

A 3D model was created on a 1:1 scale, all the dimensions for the modelling were taken from the above example. The Air confuser was modelled as a conical pipe passing through the float from top to bottom with a top diameter of 150 (mm) and a bottom diameter of 100 (mm). The same boundary conditions were assigned for both solvers: a water inlet was applied by means of a velocity inlet- 0.262 (m/s) to the cylindrical shape surface between the float and funnel crest; the air inlet was applied by means of opening to the top surface of the confuser with atmospheric pressure; outlet was applied by means of the outlet with atmospheric pressure at the end of the outflow tract; all the walls were assigned as solid walls.

The ANSYS CFX input was the following: a program-controlled linear CFD fluent mesh was assigned with a max. element size 90 (mm) growth rate 1.2, an additional inflation of 12 layers with a first layer thickness of 5.656 (mm) and a growth rate of 1.3 was applied to the walls of the water intake structure. Total mesh elements 962189. k- ϵ model were chosen for calculations.

The ANSYS FLUENT input was the following: the CFD fluent mesh was assigned with a max. element size 40 mm. growth rate 1.2, an additional inflation of 5 layers with a smooth transition, and a growth rate of 1.01 was applied to the walls of the water intake structure. The total mesh elements were 4968387. A realizable k- ϵ model, with enhanced wall treatment, was chosen for the calculations. Calculations were done in pressure based model in steady mode.

RESULTS

The ANSYS FLUENT and ANSYS CFX solvers provided very close results.

Comparing the water velocity diagram of the two models, no vortex formation was observed at the funnel crest. No signif-

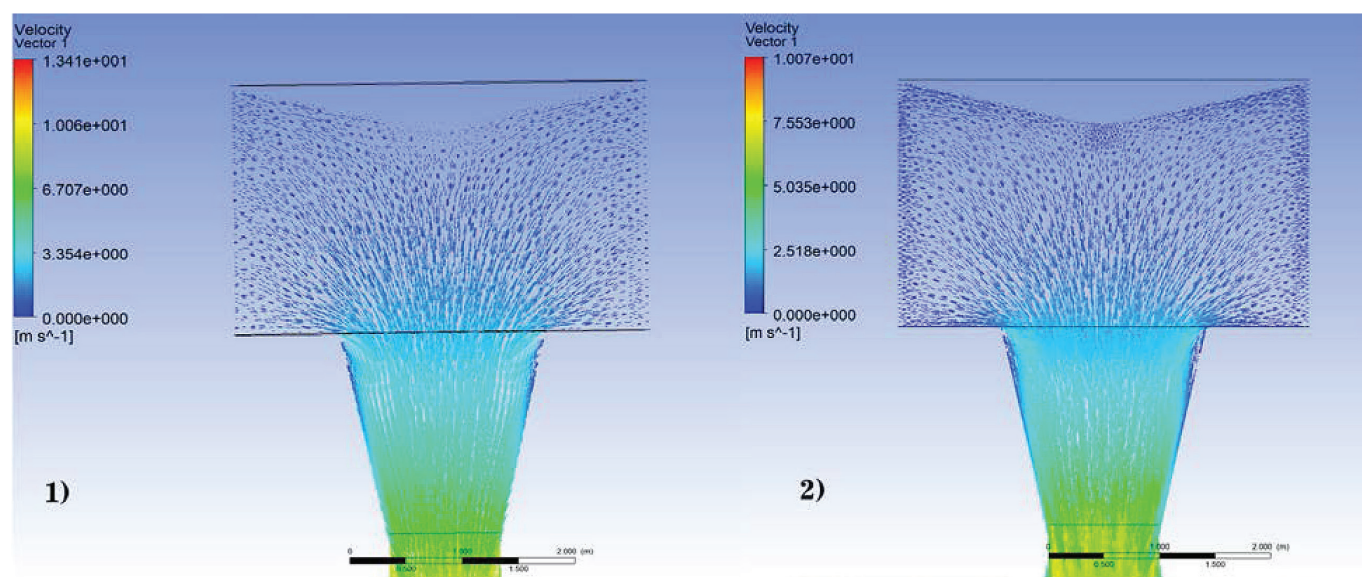


Fig. 3 Diagram of the distribution of the velocities in the longitudinal section of the funnel: 1- Ansys CFX results; 2- Ansys Fluent results

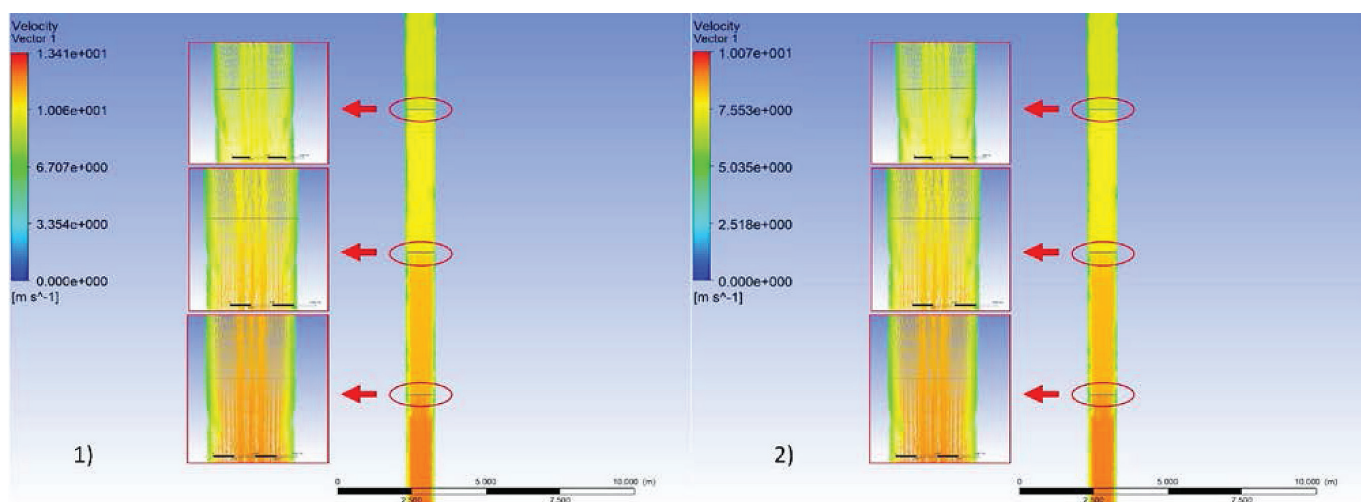


Fig. 4 Diagram of the distribution of the water velocities in the longitudinal section of the telescopic column: 1- Ansys CFX results; 2- Ansys Fluent results

icant cavitation was observed on the funnel walls. The shape of the funnel, which was obtained by analytical calculations, using the flow rate coefficient $= 0.36$, and $h_{fun}=D$ dimension ratio, enabled avoiding formation of vortex at the immersion depth calculated according to formula (1). The shape of the funnel enabled avoiding active cavitation on the walls of the funnel. Hence, a long service life and normal operation of the funnel are ensured.

structural solutions. At the same time, calculation of the curvature radius by means of equation (7) permits keeping dimensions of the elbow compact.

A velocity distribution diagram was obtained for all the main cross-sections of the water intake tract (Fig. 6).

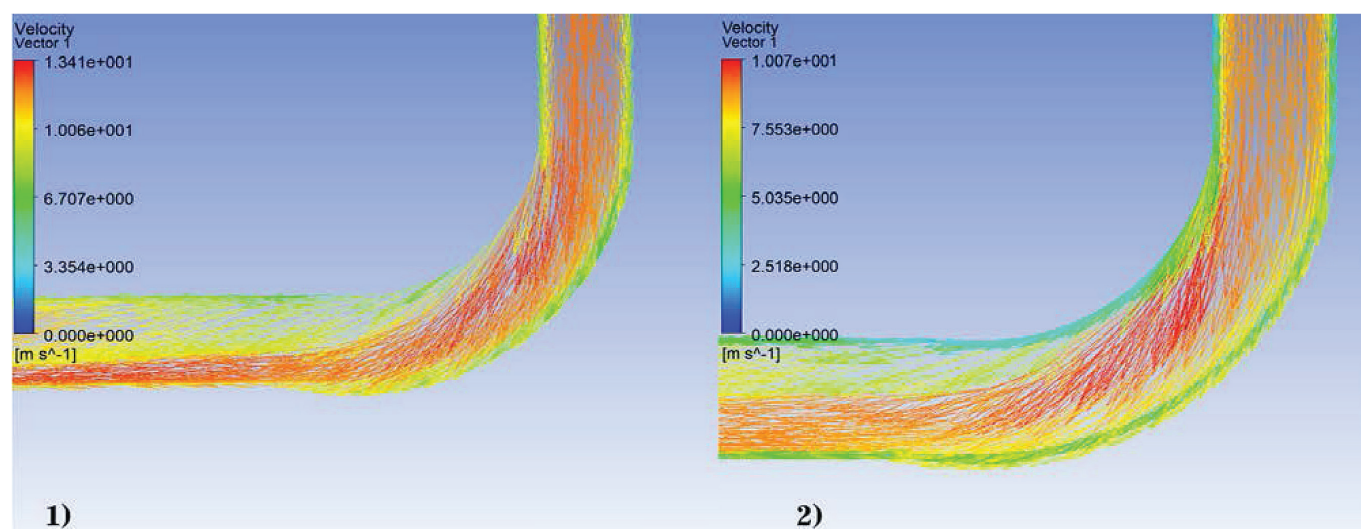


Fig. 5 Diagram of the distribution of the water velocities in the longitudinal section of the elbow: 1- Ansys CFX results; 2- Ansys Fluent results

When comparing the results of the two models, a weak turbulence of the flow without active cavitation was observed in a telescopic column in places where the cross-sections of the pipe change (Fig. 4.). Consequently, equation (6) was used for calculation of the telescopic column diameters to avoid formation of a destructive cavitation.

We observed the flow separation at the elbow. In spite of the higher velocities near the convex surface than those near the concave surface, active cavitation processes and vortex formation were not observed in the elbow (Fig. 5). The curvature radius determined by formula (7) avoids the formation of active cavitation without the implementation of additional

DISCUSSION

Since the telescopic intake takes water from the surface, the main purpose of this work was to calculate the optimal immersion depth and the shape of the inlet funnel to avoid vortex processes and cavitation in the funnel, as well as to prevent cavitation in other parts of the structure. In a previous research (Gasanov and Lipin 2021), a calculation of the telescopic water intake was presented without taking into account the critical depth of immersion and the formation of cavitation. These processes have a serious impact on the operation of the telescopic water intake and cannot be ignored.

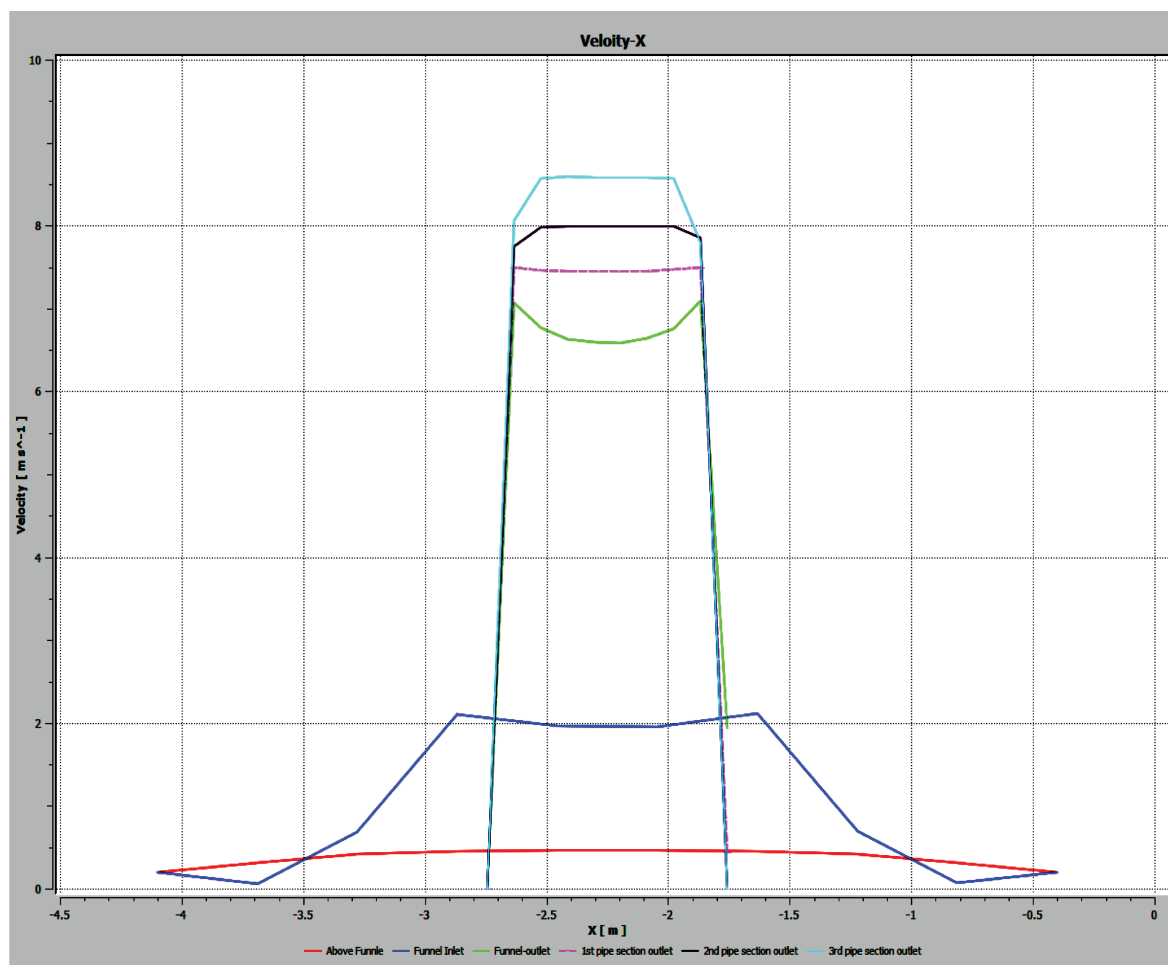


Fig. 6 Velocity distribution diagram for all the main cross-sections of the water intake track

Vortex with a gas core significantly reduces the throughput of the intake funnel and negatively impact the buoyancy of the float. To prevent vortex formation, it was necessary to determine the optimal immersion depth and the funnel shape. This shape of the inlet funnel avoids cavitation on its walls.

The results of the analytical calculations of the telescopic water intake are compared with the results of numerical models. The results of numerical models confirm the correctness of the analytical method of calculation. The results of this research helped to form a method for hydraulic calculation of the telescopic water intake.

CONCLUSION

In this work, calculations of a vertical telescopic water intake were studied. The following main conclusions were obtained:

1. A hydraulic calculation method of the telescopic water intake was developed. All the structural elements of the telescopic water intake were analyzed separately by means of existing hydraulic principles. A specific example for its implementation was given and verified by numerical calculations.
2. The equation for the calculation of the critical submergence depth of the funnel crest was proposed. The re-

sults of the manual calculations were verified by numerical modelling. At the critical submergence depth $H=1.5 D$, no vortex formation was observed at the funnel crest.

3. The optimal range of the flow rate coefficient $\mu = 0.30-0.45$ for the funnel throughput calculation is directly affecting the shape and submergence of the funnel. The shape of the funnel achieved, with proposed flow rate coefficient range and dimension ratio of the funnel $h_{fun} = D$, helps to achieve critical submergence depth $H=1.5 D$, without vortex formation. In addition, this shape protects the funnel walls from significant cavitation and erosion. The results of the numerical modelling illustrated the lack vortex and the lack of active cavitation on the funnel walls.
4. Based on the results of the numerical calculations it can be concluded, that the equation proposed for calculation of telescopic pipe diameters allows avoiding active cavitation in the telescopic column, hence protecting it from erosion.
5. An optimal elbow curvature radius $R_0 = 2.5d_{elbow}$ was proposed. The results of the numerical calculations of the model created with the above-mentioned radius showed a lack of significant cavitation and a lack of vortex. Hence, the proposed curvature radius protects the elbow from erosion without the installation of additional elements such as a vortex chamber, etc.

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EFFECTS OF COARSE AGGREGATE RAP CONTENT ON THE MECHANICAL AND SHEAR PERFORMANCE OF HMA BY THE UPT METHOD

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Abstract

In recent years, the use of materials derived from recycled asphalt has attracted the attention of airfield and highway construction researchers. Since the making of these products is economically cost-effective, one idea about how to use them is to use coarse-grained crumbs from recycled Asphalt Pavement (RAP), which are agglomerates that can be found in coarse RAP, as a replacement for virgin asphalt. The goal of the current study is to investigate the performance of hot mix asphalt created with various percents of RAP. The punching shear strength of cylindrical specimens versus the penetration of steel rods with fixed and determined diameters into an asphalt specimen is one of the tests used to study the shear strength of asphalt. Other typical tests include a test of the structural strength and resilient modulus with two rectangular and sinusoidal loading shapes, respectively, as well as the specimen's strength against different temperatures of asphalt mixtures containing 10, 20, and 30% coarse-grained RAP. The results show that the resilient strength of the samples with a maximum 30% coarse-grained RAP is within reasonable limits. Furthermore, the semi-sinusoidal loading has a higher resilience modulus than rectangular loading. It is also observed during uniaxial penetration testing (UPT) that increasing the RAP increases the penetration of the steel rod, thereby implying that the shear strength of the asphalt has decreased.

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Key words

- Sustainable development,
- Reclaimed Asphalt Pavement (RAP),
- Uniaxial shear-strength penetration,
- Airfield and road pavements.

1 INTRODUCTION

Environmental compatibility and cost-effectiveness are currently two of the most relevant project requirements. The quality of the materials used in a road pavement is critical in its design as well as in the operational specifications of the roads. In cases where the materials desired are not available or are located very far from the construction site, the feasibility of using materials that are available seems necessary. If Recycled Asphalt Pavement (RAP) can be used instead of natural materials while preserving or enhancing a pavement's efficiency,

that would be a positive move towards improving sustainable environmental practices and lowering road pavement costs (Singh *et al.*, 2018). Furthermore, in order to move towards sustainable regional development, operational costs can be minimized by introducing new methods for fundamental road improvements and recycling weakened asphalt layers in roads and airports, as well as improving the bearing capacity of road structures (Gao *et al.*, 2018). With this method, the use of natural stone mines in improvement projects will, on the one hand, be reduced (Nosetti *et al.*, 2018). On the other hand, a large part of harmful materials and waste, which could endanger the

environment and undermine a natural landscape, can be recycled and reused (Prabhu, *et al.*, 2019). Various researchers have conducted strength tests on asphalt mixtures using virgin bitumen and aggregates as well as recycled materials (Ghavibazoo, *et al.*, 2017) and have investigated the effect of rejuvenating agents on mixtures containing a high percentage of reclaimed asphalt pavement (Ismael, Khaled, 2018). Another topic that has piqued the interest of researchers is the use of hot asphalt additives (organic, chemical, and synthetic and natural zeolite) in asphalt mixtures (Sengoz, *et al.*, 2017). Various types of aggregates, such as slag aggregates, have been used in hot asphalt mixtures, for example, to help with environmental issues (Falah Tafti, *et al.* 2017). It has been demonstrated that increasing the RAP content in an asphalt mix has a significant impact on the viscoelastic properties of the blend of the binder recovered from the RAP and the virgin bitumen (Omranian, *et al.*, 2018).

The dynamic creep test is one of many methods used by researchers to determine the optimum percentage of RAP in asphalt mixtures. The results of this test show that by increasing the percentage of RAP, the flow decreases (Jomoor, *et al.*, 2019). Another test that has been used by researchers is the punching shear stress test for cylindrical specimens, which is related to the strength of the specimens against the loading. Choosing this method is suitable because it can be performed with ordinary loading devices with pistons and rods. The strength of the asphalt compared to the penetration of a steel rod with a fixed diameter into the specimen is determined in this special test (Chen, *et al.*, 2006).

In other words, the amount of penetration of a rod with a fixed diameter into cylindrical specimens (as specified by ASTM D6927) is calculated in terms of the amount of the loading. Variables in this test include the loading rate and how it is applied (statically and/or dynamically), as well as the diameter of the rod (Faruk, *et al.*, 2015). In these tests, the force method is applied is similar to the uniaxial compressive strength (UCS) test of the asphalt (Basu, *et al.* 2019).

The current study aims to investigate the use of dynamic loading tests in addition to classic tests for asphalt mixtures containing different RAP percentages, as well as the UCS test, as a result of the penetration of the external object in terms of the amount of RAP materials consumed. This is demonstrated by giving the reduction in the amount of fresh bitumen used in the specimens, including recycled materials. The UPT test can be an alternative to other current tests due to fixing the sample and static or dynamic loading in a simpler way. In addition, the diagonal failure in the current Marshal Test is also not a direct measurement of a major property of asphalt mixtures. Based on a review of previous studies and research, it has been determined that until now, the recognition of the performance of recycled asphalt by the UPT method has not been undertaken. Because various stress levels are applied to recycled asphalt during its use, it is necessary to use these different stress levels during loading.

2 RESEARCH METHOD AND TESTING

The purpose of this research is not only to introduce the not widely known UPT laboratory method, but also to deal with the effects of using various loading conditions in asphalt mixtures containing different percentages of RAP. Therefore, two

different forms of adjustable loading by the Servo-pneumatic universal testing machine (UTM) loading device, including rectangular and semi-sinusoidal loads at different intensities, were chosen, considering that all types of vehicles load a pavement at different speeds. Evaluating effective parameters in increasing the shear resistance of asphalt mixtures can help asphalt mixtures to be designed with better performance. In asphalt mixtures containing recycled asphalt, failure can occur in specific shear modes, pure tension, shear with compression, or shear with tension. In addition to the effect of the bitumen and RAP percentages, the temperature, and different design mixing parameters, simultaneously with the occurrence of other factors such as the loading condition and stress level, can affect the elastic behavior of asphalt mixtures. These factors result in a lot of complications in the behavior of asphalt, so considering different parameters will make the research more difficult and complicated; therefore, the study has other goals besides the important ones listed, such as;

- Determination of the physical and mechanical characteristics (resilient modulus and Marshal stability) of asphalt materials containing RAP
- Determination of different loading factors and amounts of raw materials on the shear behavior of asphalt containing RAP
- Feasibility of performing the shear strength test by the UPT method for an ordinary asphalt mixture or one containing RAP

2.1 Materials

Aggregates constitute most of the ingredients of an asphalt mixture. Aggregates play an important role in the quality of an asphalt mix. In the present study, as seen in Table (1), aggregates have acceptable technical specifications. The grading of the aggregates shown in Fig. (1) according to pavement international standards is appropriate for surface and binder courses. In this diagram, the upper and lower limits of the grading curve and the average limit are displayed.

Tab. 1 Physical specifications of coarse-grained aggregates

Specification	Standard	Results	Regulation limits
Coarse aggregates			
Actual Specific Weight (g/cm ³)	ASTM C127	2.594	-
Specific gravity (g/cm ³)	ASTM C127	2.672	-
Water absorption (%)	ASTM C127	0.6	2.5 Max
Los Angeles wear (%)	ASTM C131	17.1	25 Max
Weight loss with sodium sulfate (%)	ASTM C88	3.6	8 Max
Fracture Value (%)	ASTM D5821	97	90 Min
Fine aggregates			
Specific Weight (g/cm ³)	ASTM C127	2.689	-
Specific gravity (g/cm ³)	ASTM C127	2.734	-

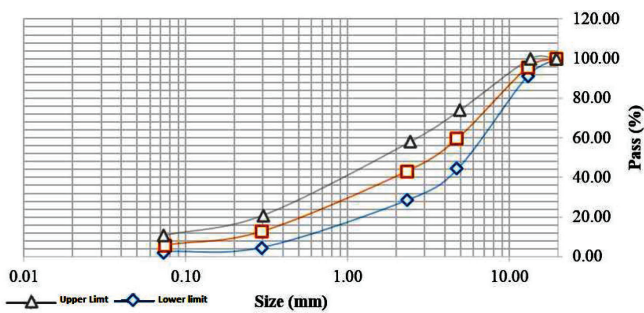


Fig. 1 Particle size distribution of aggregates with upper and lower limits

During pavement repairs on the Yazd-Nain highway in Iran, RAP materials obtained from an asphalt milling machine were deposited on the roadside. As shown in Fig. (2), the image of the depot, the initial form of the RAP, and the crushed material can be seen, as well as the aggregates after being separated from the bitumen. The aggregate grading diagram is shown in Fig. (3).



Fig. 2 (a) The location of the extraction of the crushed asphalt materials, (b) The appearance of the coarse-grained RAP after eliminating the bitumen.

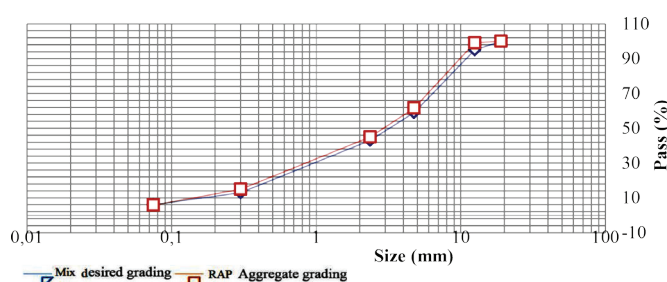


Fig. 3 Aggregate grading diagram for the optimal graded asphalt.

RAP is graded into four categories: smaller than 4.75 mm, 4.75-12.5 mm, 12-19.5 mm, and larger than 19 mm; aggregates larger than 19.5 mm and smaller than 4.75 mm were eliminated from the extraction process. These sieves and sizes were chosen because they are more common in asphalt factories. The aggregates were mixed with raw stone materials in a mixture ratio of zero (control sample), 20%, and 30%, so that the desired grading could be provided according to the regulations and guidelines, as seen in Table 1. In this study, the recycled materials were used in crushed form and, according to the results of the tests on the state of the compacted asphalt before cutting and crushing, the amount of the air voids was 2.14%, and the asphalt density was 2330 kg/m³.

The bitumen used in this study is PG 64-22, which is provided by the Jey Oil refinery in Iran; its specifications are shown in Table 2. The characteristics are based on the reports of the laboratory of the Jey Oil Refinery Company.

2.2 Mix design and preparation of samples

Different percentages of coarse-grained RAP aggregates were added to the raw aggregates, and the mixing scheme was carried out using Marshall and ASTM 1559; this method has been used by other researchers (Prathyusha, Shivananda, 2018). A total of more than 50 asphalt test specimens were used for the multiple tests. For coding the specimens, the RAP percentage and the bitumen percentages were shown as “R” and “B”, respectively. For example, R0B3.5 shows a sample with 0% RAP and 3.5% virgin bitumen. Table 3 shows the samples tested in this study. As the bitumen in the RAP is aged, virgin bitumen should be used, and the more RAP material used, the less new bitumen is used (see Table 3). The maximum temperature for mixing the RAP materials was 100°C. They were such as hot aggregate bin feeders. The temperature of the raw aggregate material was 180°C. The mixing was done at a specified weight ratio. Because the flow of the asphalt samples affects rutting, the Marshall stability test was performed for the samples after performing the typical asphalt mix tests.

2.3 Resilient modulus and structural strength tests

The results from determining the elasticity modulus of the asphalt mix samples were used in designing the thickness of the asphalt layer. One of the factors that can influence the structural strength results determined is the configuration of the loading (variations in loading over time). In this study, two types of loading conditions with varying loading intensities were used: semi-sinusoidal and rectangular loadings (Figure 4). Since pavement loading is done in different pulse forms and at different frequencies, a different loading form was applied in this study to the pavement; moreover, the rectangular waveform occurs in the lower parts of the asphalt layer in the real pavement cross-section. The shape of the wheel-pavement contact surface, speed, wheel load, depth and thickness of the pavement layer, and surface temperature are some of the variables that affect the intensity and pulse form of loading and can have different conditions. In semi-sinusoidal loading mode, by increasing and decreasing the forces on the sample, horizontal deformations are increased and reduced correspondingly. In rectangular loading, the rate of the increasing load

Tab. 2 The results of the operational tests of the bitumen used in the present research.

Test Title	Unit	Result	Standard value	
			Min	Max
Humidity	%	18	---	---
Viscosity @ 135 (RV)	°C	0.332	---	3
Flash Point Temperature	°C	334	230	---
Original Dynamic Modulus @10 rad/sec (AASHTOT315)	kPa	1	1.14	---
RTFO, Percent Change of Mass	%	5.7	---	1
MSCR Test, Standard traffic"s", $J_{nr3.2}$	kPa ⁻¹	3.9	---	4.5
MSCR Test, Standard traffic"s", J_{nr} diff %	%	19	---	75
RTFO, Dynamic Modulus	kPa	2.31	2.2	---
PAV, Dynamic Modulus @25°C	kPa	1632	---	5000
PAV, Creep Stiffness @-12°C	MPa	96.23	---	300
m-value, (slope) @-12°C	No unit	0.302	0.3	---
PAV, Direct Tension	%	---	1	---

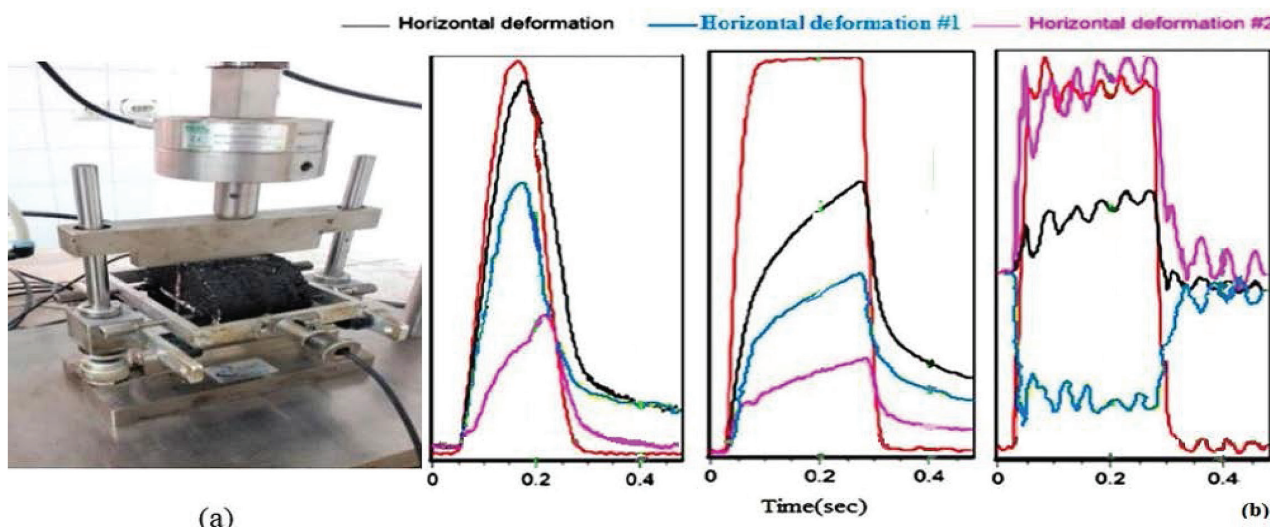
Tab. 3 The samples used in this research

Name of sample	RAP (%)	Virgin Bitumen(%)
R0-B3.5	0	3.5
R0-B4	0	4
R10-B3	10	3
R10-B3.5	10	3.5
R20-B2.5	20	2.5
R20-B3	20	3
R30-B2	30	2
R30-B2.5	30	2.5

and deformation does not match. The plastic deformation of the rectangular loading is more than the semi-sinusoidal.

2.4 Equipment and approach to test the uniaxial penetration

Investigating the shear strength of asphalt is important because grooves appear on many roads after passing traffic due to a lack of shear strength in the asphalt mixture. The punching shear test is one method for determining shear strength. Reducing the shear strength and/or increasing the probability of the rutting phenomenon in the asphalt including RAP are necessary for performing this test (Jomoor, et al., 2019), (Faruk, et al., 2015). The results of this test measure the shear resistance of asphalt samples at the desired temperature, and of course, usual asphalt behaviors such as rutting and fatigue are

**Fig. 4** (a) The location of the extraction of the crushed asphalt materials, (b) The appearance of the coarse-grained RAP after eliminating the bitumen.

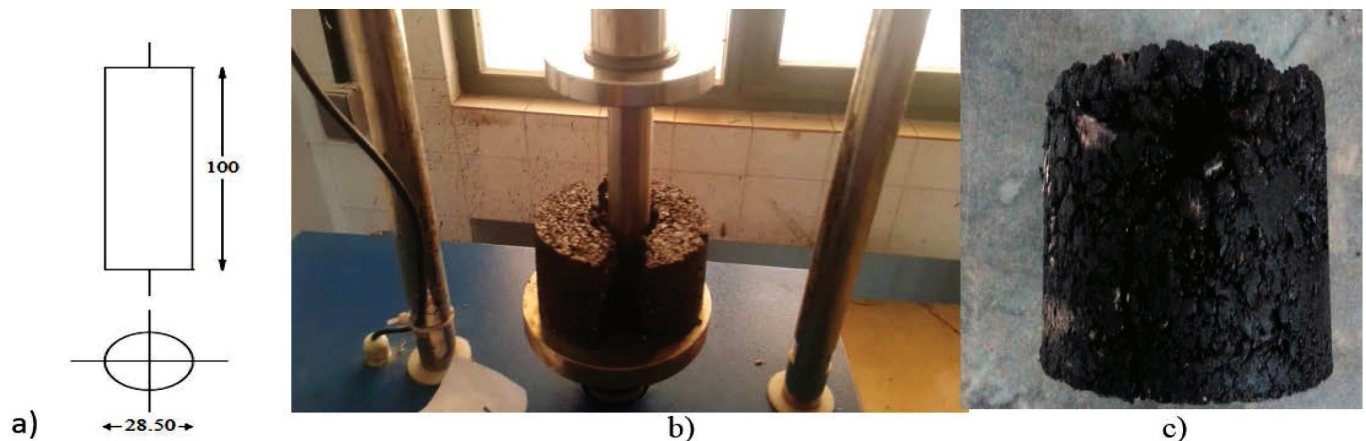


Fig. 5 a) schematic of the device, b) the asphalt sample under loading in the shear uniaxial compressive strength test, c) the sample after loading.

not directly determined. However, it is possible to obtain a relation between these parameters in supplementary studies. In this test, the samples are first warmed for 4 hours at temperatures of 40°C and 60°C; then the samples are subjected to a shaft (piston) penetration test. For loading, this was done at a speed according to the standards of Marshall's test, as shown in Fig. (5). Although this test is not the usual method of measuring asphalt performance, it can provide some characteristics of an asphalt mixture faster and more easily.

The amount of the shear uniaxial compressive strength is calculated using Eq. (1):

$$\zeta = K \frac{P}{A} \quad (1)$$

where ζ is the shear tensile of the asphalt mixture; P is the penetration force of the piston (kN); A is the area of force applied (mm^2); and K is the shape factor of the loading rod (without the unit).

Referring to the research references and given the diameter and the stainless steel rod material, the value of K was considered to be 0.327 (Faruk, et al., 2015). The penetration against the counterforce can be plotted by recording the force and amount of the rod's penetration into the asphalt specimen in the failure path. Due to the defined scope of this research, the focus of this study was on the general performance of the asphalt mixture using the UPT method; It is therefore suggested that in future studies, the characteristics of bitumen containing aged bitumen and virgin bitumen should be discussed according to the amount and importance of the behavior of aged bitumen in RAP.

3. RESULTS AND DISCUSSION

3.1 Marshall stability test

The main purpose of this paper was to investigate the changes in the asphalt mixture's properties after adding RAP. The tests were performed using a standard Marshall apparatus. The stability, flow, specific gravity and air voids of the prepared samples were recorded. As the percent of bitumen increased, the air voids in the samples decreased. The average values of the Marshall test parameters with the various percentages of bitumen are summarized in Table 4. The results of the Marshall strength test are shown in Figs. (6) and (7).

Also, the flow variations of the asphalt samples for the different percentages are shown in Fig. (5).

The results of the asphalt mix design tests show that increasing the RAP percentages reduces the amount of bitumen required. As seen in Figures 6 and 7, all the samples have adequate Marshall stability and suitable flow, so it can be seen that in the samples containing 30% RAP, the amount of virgin bitumen consumption is reduced, resulting in the consumption of 30% less natural and raw materials. Also less virgin bitumen was required. The addition of 30% RAP increased the Marshall Stability to 12%. According to figure 7, as the percent of RAP increased, so did the flow rate of the samples, reaching around 25% for 20% RAP. In other studies, a fixed bitumen percentage of 4.5% was kept constant in all the samples, and the amount of bitumen added varied as the RAP percentage changed (Sariani, et al., 2019).

Tab. 4 The average Marshall test results

Control sample	Dry weight	Weight in water	Laboratory sample volume	Specific gravity (Gmb)	Marshall stability	Flow	Air voids
	g	g	cm^3	kg/cm^3	N	mm	%
R0-B3	1201	697.5	503.5	2.39	732	3.83	3.52
R0-B3.5	1224	712.7	511.3	2.39	1138	3.8	3.94
R0-B4	1235.8	722.2	513.6	2.41	1117	3.56	4.35
R0-B4.5	1235	727.6	507.4	2.43	1018	4.04	4.79

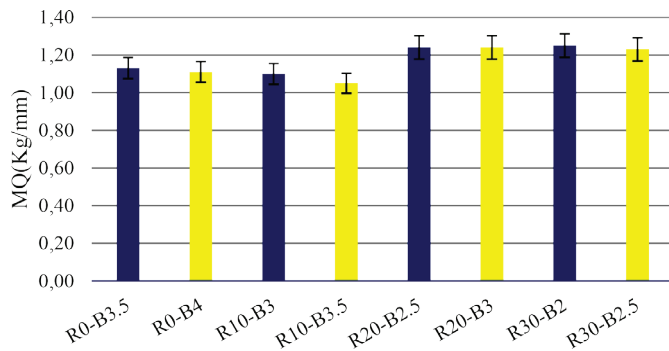


Fig. 6 Effect of RAP and bitumen percentages on the Marshall test stability

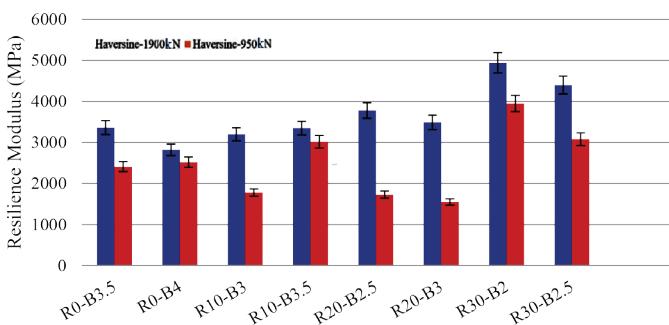


Fig. 8 Resilience modulus for semi-sinusoidal loadings with a maximum force of 950 and 1900kN (25°C).

3.2 Results of the resilience modulus and structural strength test

The resilience modulus test is one of the non-destructive tests for asphalt. In this test, due to the high sensitivity of the Universal Testing Machine device, the difference between the data and the error percentage is high. It should be noted that the height (thickness) of the samples was assumed to be fixed in this test. In a pre-test step, the amount of force that caused the asphalt samples to break and deform was obtained, and the forces selected in the test step were 1900 kN and half of this loading, in the room temperature (25°C), and the maximum temperature of the asphalt was 60 °C. The findings for the semi-sinusoidal and rectangular loadings are shown in Figs. (8) and (9), respectively.

According to Fig. (8), adding the RAP increases the resilience modulus to a maximum force of 1900kN. However, considering that not many changes were seen in the value of the resilience modulus, the addition of RAP had no discernible effect on the resilience modulus with a maximum force of 950kN. In general, the resilience modulus calculated with the maximum force 1900 kN is much greater than that calculated with the maximum force 950 kN.

The addition of RAP increases the rectangular structural strength with a maximum force of 3800 kN, as shown in Fig. (9), while the addition of RAP has no significant effect on the modulus of elasticity at a maximum force of 950 kN. But given the results shown in the plot and also given that the error in the results is less in the maximum load presented, it can be asserted that by the addition of RAP, the structural strength

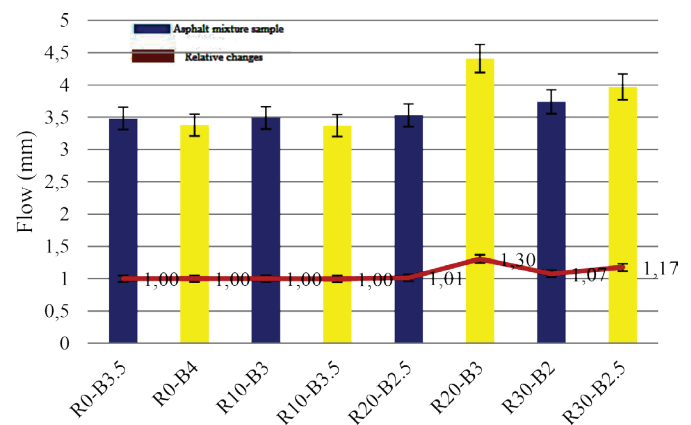


Fig. 7 Variations of the different percentages of the RAP on the flow of the asphalt samples

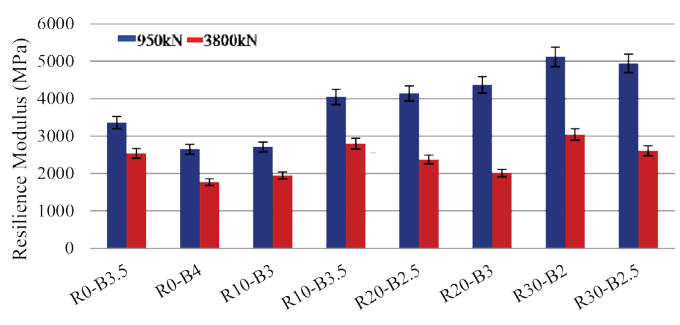


Fig. 9 Structural strength for rectangular loadings with maximum forces of 950kN and 3800kN (25°C).

for the rectangular loading with the maximum force 950 kN increases. Generally, the resistance of the structure obtained in rectangular loading, and semi-sinusoidal loading, for the maximum force of 3800 kN, is much higher than the resistance of 950 kN. In this study, the samples with 30% RAP have a higher resilience modulus than the control samples and with a lower RAP in various situations (a resilience modulus with a maximum force and different loads). As the research results indicate, an improvement in the stability values was observed with the increase in the RAP content of 25% and 50% (Al-shamleh, et al., 2022). As can be seen in Figs. (8) and (9), while the normal increase in the resilience modulus and structural strength with the increasing RAP percent is not visible, it can be stated that the 950 kN loading significantly reduces the accuracy of the resilience modulus calculated, making it unacceptable and unreliable.

3.3 Results of the uniaxial penetration test

The samples were subjected to a uniaxial shear test under a rigid steel rod penetration. In the tests, the samples were heated in the furnace for 4 hours at two different temperatures, i.e., 25°C for the first set of samples (room temperature) and 60°C for the second set of samples (the Marshall test temperature). The shear strength of the samples is shown in Figs. (10) and (11).

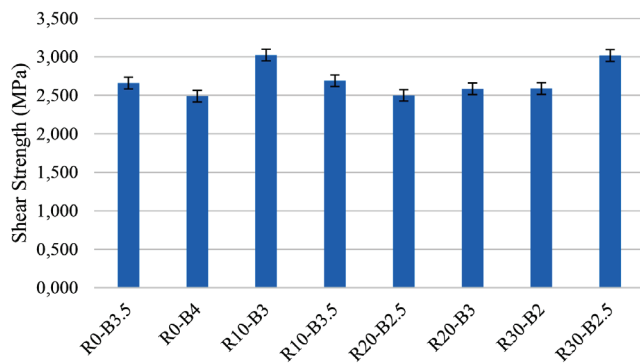


Fig. 10 Uniaxial shear strength of the different samples at 25°C.

According to Fig. (10), there is no relationship between the different sets of data related to the samples with varying percentages of the RAP. As a result, it can therefore be stated that at relatively low temperatures, the RAP has little effect on the uniaxial shear strength of the samples. The reason behind this may be that at this temperature (25°C), there is no difference between the adhesive property of the virgin bitumen added to the asphalt compared with the aged bitumen that exists in the RAP. The role of the temperature on the adhesive behavior between the aggregate bitumen and the contact surfaces between the aggregates is known, but the temperature sensitivity of the virgin and aged bitumen is different, and the sensitivity to heat is higher in the fresh bitumen.

As can be seen in Fig. (11), at a high temperature the uniaxial shear strength of the samples with RAP, is lower than that of the control samples and the samples with less RAP. In other words, with the addition of RAP to the asphalt mix variants, their uniaxial shear strength at high temperatures decreased. In previous research, the reduction in the shear strength of the samples having RAP materials has been widely reported (Ismael, Khaled, 2018). The reason for this phenomenon could be that the properties of the old bitumen inside the mastic of the RAP change over time (from the initial asphalt production to the road extraction and then the reproduction of the asphalt mix). Therefore, the fresh bitumen has less adhesion at a high temperature, as seen in the results of other researchers (Zadshir, *et al.*, 2018). This phenomenon is the primary weakness of samples with coarse-grained RAP. It should be noted, however, that using these materials in cold areas would not be technically problematic. In addition to the shear strength of the samples, the penetration of the steel rigid rod into the samples can also provide useful results; for instance, the quantity of the penetration depends on the shear stability of the asphalt specimens. Therefore, the depth of the rod penetration into the samples at different temperatures is shown in Fig. (12).

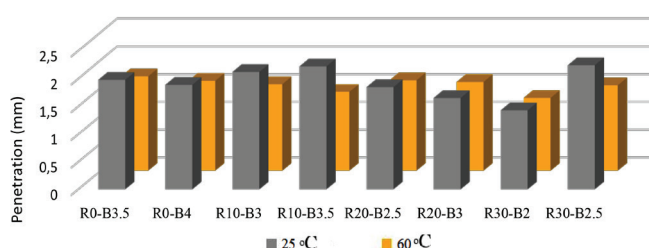


Fig. 12 Penetration of the rod into asphalt samples at various temperatures.

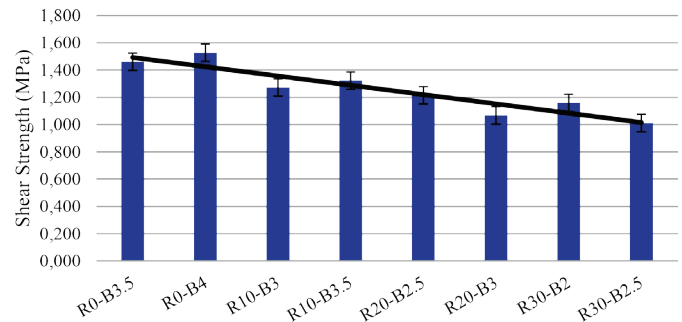


Fig. 11 Uniaxial shear strength of the different samples at 60°C.

According to Fig. (12), increasing the temperature reduces the permeability of the steel rod. This means that as the temperature rises, the samples fail sooner and do not achieve the penetration rate in the test mode at lower temperatures, thereby resulting in more deformable mixtures.

4 CONCLUSION

The aim of the present study was to investigate the operation of hot mix asphalt produced with different percentages of recycled asphalt pavement. The punching shear strength of cylindrical specimens was used to determine the shear strength of the asphalt in this study. The other common test was the resilient modulus test with two loading forms. The tests were done for samples with 0, 10, 20 and 30% RAP. The results were reported as follows:

- Recycled materials could be used to reduce consumption of materials, save financial resources, protect the environment, and adhere to the principle of sustainable development. This could cut the cost of providing the necessary materials by up to 30%.
- When different percentages of RAP were added, the strength of the samples was acceptable when compared to the control samples. When the recycled RAP was added, the flow of the asphalt samples increased. The greatest increase was 25%, which was related to the sample with 20% RAP.
- The resilience modulus of the RAP samples is about 10-50 percent higher than the control samples, and the resilience modulus in a semi-sinusoidal loading scenario was higher than the structural strength in the rectangular loading scenario. The difference between the values in these two loading modes increased as the RAP content increased.
- At low temperatures, the shear strength of the RAP samples and the control samples were nearly equal; however, heating the samples reduced the strength of the RAP samples even more. This reduction could reach 66% in samples containing 30 % RAP. Furthermore, increasing the RAP percentages increased the penetration of the rigid rod into the samples.
- Increasing the temperature reduced the penetration rate of the rigid rod into the samples. This decrease was greater in the RAP-containing samples. By increasing the temperature in the sample with 10% RAP, the penetration decreased by about 28%.

This study focused on the shear stresses that play a major role in the performance of RAP, but in order to control the other functional parameters of asphalt mixtures, it is necessary to meet the standards. It is suggested to continue this study by conducting more experiments to achieve regression equations between the variables measured. Due to the defined scope of this research, the focus of this study was on the general performance of an asphalt mixture using the UPT method; it is therefore suggested that in future studies, according to the amount and importance of the behavior of the aged bitumen in RAP, the characteristics of bitumen containing aged bitumen and virgin bitumen should be investigated and discussed.

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