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Original scientific paper

Potential of Portland pozzolana cement in the stabilization of an expansive soil subjected to alternate cycles of wetting and drying

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ABSTRACT

Cement/lime stabilization of soils is one of the common techniques adopted for improving its geotechnical properties. Lately, the focus of investigation has shifted to blended stabilization with industrial wastes as auxiliary additives. However, the role of blended cement in stabilization of soil has been studied insufficiently despite the fact that it is manufactured under controlled conditions. This investigation deals with the use of Portland pozzolana cement (PPC) instead of ordinary Portland cement (OPC) in the stabilization of an expansive soil subjected to alternate cycles of wetting and drying. Unconfined compression strength (UCS) test specimens of dimensions 38mm x 76mm were cast and cured for periods of 7, 14 and 21 days. Then, the specimens were subjected to 1, 2 and 3 cycles of wetting and drying and the UCS of the specimens were determined. Based on the results of the investigation, it was found that OPC performed significantly better than PPC under normal conditions. However, under conditions of wetting and drying, PPC stabilized soil performed much better than OPC stabilized soil when sufficient binder content was available.

1 Introduction

Expansive soils are well known for their detrimental volume change behaviour and the resulting effects on structures built on them. They are especially dangerous to lightly loaded structures due to the immense swell pressure generated by such soils when they imbibe moisture. Stabilization of such soils have been practiced for quite a while now to mitigate their devastating effects on the structures built on them. The most commonly adopted stabilization technique for such soils is chemical stabilization using either lime or cement. Cement stabilization, however, is not that effective in the case of extremely plastic swelling clays [1]. The use of industrial wastes as auxiliary additives to cement for improving its potential is well documented [2]. But it is a well-known fact that manufacture of cement has a very heavy carbon footprint. There has been extensive research in reducing this carbon footprint of cement use in the construction industry and soil engineering with no exception. The potential ways available are (1) Development of an alternative binder to cement (2) Partial replacement of cement with supplementary cementitious materials/pozzolans. There has been few studies going in the development of an alternative binder [3]–[6]. But the mainstream popularity of ordinary Portland cement (OPC) is yet to be challenged. On the other hand, partial replacement of cement seems to be rather successful with several materials identified for replacement. This partial replacement can be achieved at two levels. One, at the manufacturing stage where the part of the raw materials is replaced to develop blended cements. Two, at the application stage, when a part of the cement content for the required application is replaced with supplementary materials. In the area of soil stabilization, the latter method of partial replacement of cement with supplementary materials especially solid wastes is quite popular. The former technique of blended cements like Portland pozzolana cement and Portland slag cement have slowly started to gain acceptance, especially in the area of concrete technology. However, in the field of soil stabilization, use of blended cements in stabilization has not become as popular as the use of Portland cement. This, despite the fact that partial replacement of cement with supplementary materials at the field level has been as successful as in soil stabilization. A sift through literature reveals the fact that there have not been that many investigations involving use of Portland pozzolana cement (PPC) in the stabilization of soil. Patowary et al. [7] investigated the use of PPC in the development of

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stabilized soil blocks. Patel et al. [8] investigated the bearing performance of PPC stabilized soil. Barman and Das [9] investigated the performance of two different types of soils stabilized with PPC. Cassiophea [10] investigated the combination of PPC and dolomite in stabilization of a clayey subgrade. Patel et al. [11] investigated the effects of variable dynamic compaction on the PPC stabilization of clayey soil. The available literature on stabilization of soils predominantly focusses on OPC rather than PPC. The few investigations carried out on PPC look into the fundamental properties of the stabilized soils. There exists a need to investigate the potential of PPC in soil stabilization for varying conditions of durability which are encountered in the field. Thus, this investigation attempts to evaluate the potential of PPC in stabilization of an expansive soil and compare it with the performance of OPC under normal conditions as well as conditions of alternate wetting and drying.

2 Materials

Soil deposited near a lake in Thaiyur in Kalavakkam, Tamil Nadu, was collected and characterized in the laboratory for its geotechnical properties. Table 1 shows the various properties of the soil as determined in the laboratory based on various codes of the Bureau of Indian Standards (BIS). The maximum size of the soil particles was not greater than 2 mm. In fact, 97% of the soil was finer than 75-micron sieve as seen from Table 1. The cements used in this investigation where commercially available OPC and PPC.

Table 1. Properties of the soil

Property	Value
Liquid limit [12]	67%
Plastic limit [12]	20.7%
Plasticity index	40.7%
Sand Content [13]	3%
Silt Content [13]	28%
Clay Content [13]	69%
Specific gravity [14]	2.67
Optimum moisture content [15]	18%
Maximum dry density [15]	16.30 kN/m^3
Unconfined compressive strength (UCS) [16]	103 kpa
Classification [17]	CH

3 Methods

The experimental methodology adopted in this investigation consisted of the following stages.

3.1 Preparation and characterization of soil

The soil sample collected from near the lake was brought to the lab and prepared for experimentation based on the procedure stated in BIS code [18]. This was followed by the geotechnical characterization of the soil in the laboratory for determination of its various properties and classification of the soil.

3.2 Selection of stabilizer content

Two trial cement contents for stabilization of the soil were fixed. The cement contents fixed were the same for both OPC and PPC for a one-to-one comparison of the results. Soil cement usually contains less than 5% cement content [1]. Thus, two contents, one below and the other above 5% were selected for the investigation. In this investigation, the contents selected were 2.5% and 7.5%.

3.3 Determination of compaction characteristics

The soil stabilized with 2.5% and 7.5% contents of OPC and PPC separately were subjected to compaction tests using the mini compaction apparatus [15], in accordance with the procedure laid down in BIS code [19] for stabilized soils.The mini compaction mould is a circular mould of 3.81 cm internal diameter and height of 10 cm with a 3.50 cm removable collar. The total volume of the mould is 114 cm^3. The hammer has a height of 3.5 cm, weighs 1 kg and falls freely over a foot in contact with the soil over a height of 16 cm. 200 g of soil finer than 2 mm was taken for the test and compacted in three layers with 36 blows per layer to replicate the standard proctor test. The test was repeated for different water contents to identify the maximum dry density (MDD) and optimum moisture content (OMC) for achieving the MDD. This was done to determine the MDD and OMC values of the stabilized samples for preparing UCS specimens. BIS code [19] recommends that in the case of compaction tests for stabilized soils, separate soil samples mixed with the stabilizer with increasing water content be used for each of the trials of the compaction test instead of increasing the water content in the same soil sample as done conventionally.

3.4 Preparation and curing of specimens

UCS specimens of dimensions 38 mm x 76 mm were prepared at their MDD and OMC values by static compaction. Then, the specimens were demoulded and immediately packed in a sealable polythene cover for curing for a period of 0 (2 hours), 7, 14 and 21 days. Three samples were cast for each combination. At the end of the period of curing, the samples were strained at a rate of 0.625mm/min until shear failure to evaluate the UCS.

3.5 Simulation of wetting and drying

Separate samples were cast for simulation of wetting and drying. The samples were cured for a period of 21 days before subjecting them to cycles of wetting and drying. The samples were placed in a bed of soaking wet cotton and then covered by another layer of soaking wet cotton to simulate the wetting cycle for a period of 24 hours. Care was taken to ensure that the bed of cotton stayed wet throughout the duration of the wetting cycle. This was followed by 24 hours in open air at room temperature which constituted one cycle of drying. Similar procedure has already been reported in literature [20], [21]. The samples were subjected to 1, 2 and

3 cycles of wetting and drying. This was followed by UCS testing to evaluate the effect of wetting and drying cycles on the strength.

4 Results and discussion

The results of the investigation involving the stabilization of an expansive soil under normal as well extreme conditions using OPC and PPC is discussed in the following sub sections.The compaction characteristics of the stabilized soil were determined to prepare the UCS samples. Table 2 gives the compaction characteristics of the OPC and PPC stabilized soil specimens.

Table 2. Compaction characteristics of the stabilized specimens

Combination	MDD (kN/m^3)	OMC (%)
2.5% OPC	15.59	25.5
2.5% PPC	15.62	29.5
7.5% OPC	16.43	21
7.5% PPC	15.44	27.5

4.1 Strength of OPC and PPC stabilized soil

Figure 1 shows the development of the strength of OPC and PPC stabilized soil with increase in curing period. From the figure, it can be seen that the strength of 2.5% PPC stabilized soil after 21 days of curing was 186 kPa, up from 103 kPa for pure expansive soil. However, on stabilization using 7.5% PPC, the achieved strength was 700 kPa, which was a significant improvement. When the expansive soil was stabilized with 2.5% OPC, the strength was 386 kPa after 21 days of curing, which is more than half of what was obtained using 7.5% PPC. When the soil was stabilized using 7.5% OPC, the achieved strength after 21 days of curing was 1652 kPa, which was more than double of what was achieved by the same content of PPC. Barman and Das [9] reported a strength of 119.54 kPa and 209.85 kPa for 2% and 8% PPC stabilized clayey soil after 28 days of curing whereas James and Pandian [22] report very high early strengths of more than 2000 kPa for 2% and 3% OPC stabilization. Thus, it is clear that the strength of the stabilized soil is the lowest for 2.5% PPC stabilization and rises with the increase in the binder content to 7.5% PPC. Comparing OPC and PPC stabilization, the strengths developed by OPC was more than twice that of PPC for both the stabilizer contents. Thus, it is obvious that under normal conditions, OPC stabilization of expansive soil is much better when compared to PPC stabilization, irrespective of the binder content

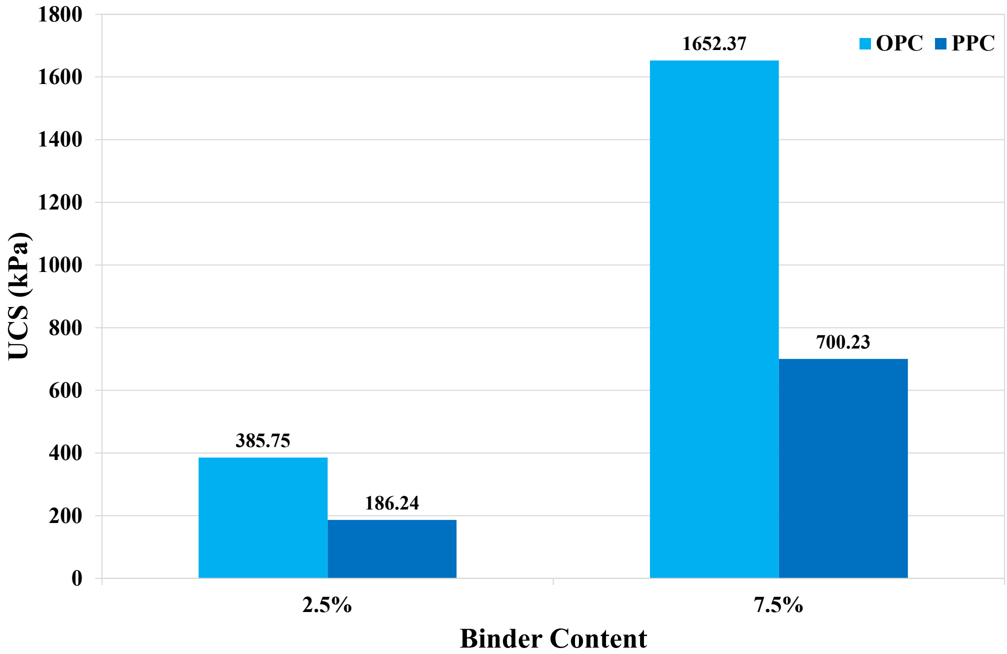

Figure 1. Strength of OPC and PPC stabilized soil at 21 days of curing

Figure 2 shows the development of the strength of the stabilized soil with curing period. From the figure, it can be seen that strength increases with curing which is a well-established fact. It can also be seen that the trends of development have good correlation with the actual data as seen from the R^2 values. However, it can be seen that development of the strength of PPC and OPC is significantly different at higher binder content. The strength development of OPC at higher binder content is significantly better when compared to PPC. At lower binder contents, the strength development seems to be similar superficially with strength gain being more or less flat when compared to stabilization at higher binder content. In order to understand this better, an analysis of the rate of strength gain of all the four combinations was done. The entire range of curing was divided into three stages viz. from 0 -7 days, 7 – 14 days and 14 – 21 days. The rate of strength gain was determined by obtaining the slope of the curve between the boundaries of each stage. A similar analysis was performed earlier by Naveena et al. [23].

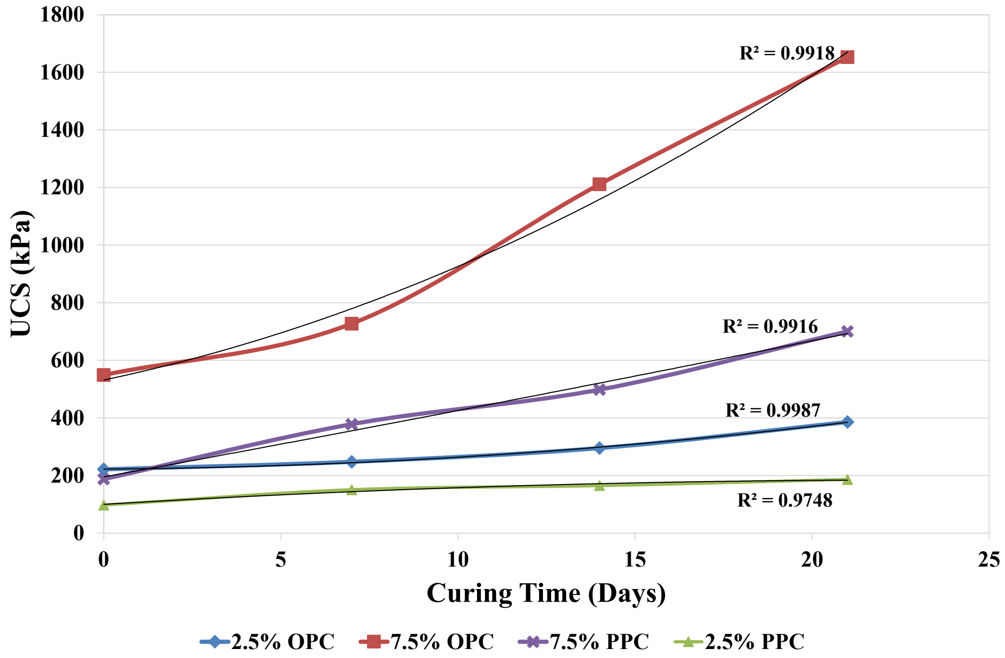

Figure 2. Development of the strength of OPC and PPC stabilized soil with curing

Figure 3 shows the rate of the strength gain for all the four combinations.To determine the strength rate gain of 7 days, the 0 day strength of all samples were determined at 2 hours of curing based on earlier literature [24], [25].

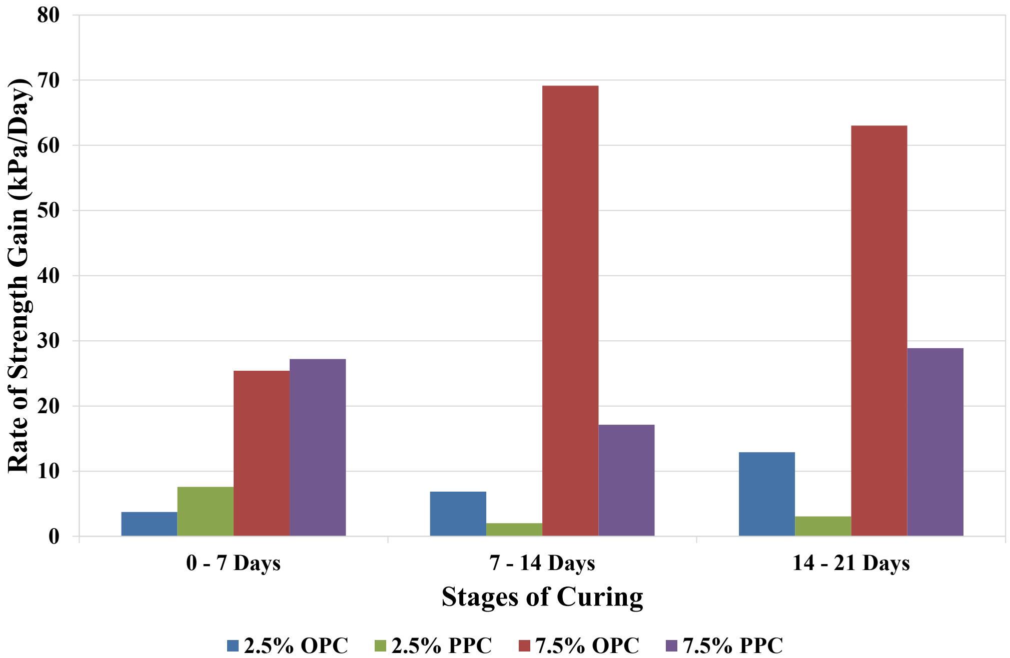

Figure 3. Rate of strength gain with curing period for OPC and PPC stabilized soil

Comparing the binder content, the rate of strength gain of PPC at lower binder content of 2.5% is 7.6 kPa/day which is double that of 2.5% OPC stabilized soil at 3.73 kPa/day. It can be seen that PPC performs better in early curing period when compared to OPC, however, it should be noted that the absolute strength of OPC stabilized soil is still higher than PPC stabilized soil at 7 days. This may be due to the higher strength developed by OPC stabilized soil at 2 hours curing when compared to PPC stabilized soil. Despite the slower rate of strength gain in the early stage, the higher starting strength of OPC stabilized soil results in the strength of OPC stabilized soil staying higher than PPC stabilized soil. However, when the binder content is increased to 7.5%, the strength gain rate of PPC and OPC stabilized soils are 27.22 and 25.42 kPa/day, respectively. Thus, at higher binder content, there is only a small difference in the strength gain rate, with the absolute strength values of OPC stabilized soil significantly higher than PPC stabilized soil. When curing time is increased, the strength development of OPC stabilized soil is significantly better than PPC stabilized soil during the second and third stage of curing, making up for the slow rate of strength gain in early curing. A similar trend was reported by Naveena et al. [23] where the rate of strength gain was 7.94 kPa/day during the first seven days when the cement content was 3%. However, when cement content was increased to 9%, the rate of strength gain increased to 93.4 kPa/day in the first seven days. The trends of strength gain rates of PPC and OPC stabilization are quite the opposite. The strength gain rates of OPC stabilization is maximum in stage 2 whereas in the case of PPC stabilization it is the lowest of the three stages. Table 3 gives the results of UCS of all combinations of stabilized soil specimens.

Table 3. Average UCS of all combinations for different curing periods and cycles

Curing	UCS (kPa)			
	2.5% OPC	2.5% PPC	7.5% OPC	7.5% PPC
0 Days	221.41	97.54	548.99	187.59
7 Days	247.51	150.71	726.97	378.17
14 Days	295.43	164.79	1211.07	498.11
21 Days + No Cycle	385.75	186.24	1652.37	700.23
21 Days + 1 Cycle	402.76	176.82	2330.34	1104
21 Days + 2 Cycles	446.9	291.43	1579.16	1664.34
21 Days + 3 Cycles	451	300.13	1590.95	1839.18

4.2 Durability of OPC and PPC stabilized soil

The durability of OPC and PPC stabilized soil was determined by subjecting the samples to alternate cycles of wetting and drying. Figure 4 shows the durability of OPC and PPC stabilized soil subjected to 1, 2 and 3 cycles of wetting and drying. From the figure, it can be seen that the performance of both OPC and PPC stabilized soil is similar at lower binder content of 2.5%. The increase in number of cycles result in a marginal increase in the strength of both the OPC and PPC stabilized specimens. The increase in strength can be attributed to the increase in time of chemical reactions with wetting and drying cycles as well as the increase in cementitious compounds during wetting and drying cycles [26]. Similar increase in the strength with the increase in number of cycles have been reported by several other researchers as well [21], [27], [28]. In the case of 2.5% OPC stabilized soil, the strength of the specimens increases from 385 kPa to 451 kPa when the number of cycles increases from 0 to 3. PPC stabilized soil also performs similarly with the strength of 2.5% binder content stabilized soil increasing in strength from 186 kPa to 300 kPa. OPC is still better than PPC at this quantum of stabilizer irrespective of no. of cycles of wetting and drying. However, the durability performance completely changes at higher contents of binder. The strength of 7.5% OPC stabilized soil increases from 1652 kPa to 2330 kPa for the first cycle of wetting and drying. On further increase in the wetting and drying cycles, the strength drops to 1579 kPa for two cycles and then stabilizes at 1591 kPa for three cycles. On the other hand, the strength of 7.5% PPC stabilized soil steadily increases from 700 kPa to 1104 kPa and 1664 kPa for one and two cycles of wetting and drying. The strength gain then flattens to 1839 kPa for three cycles of wetting and drying. Thus, at 7.5% binder content, OPC performs better than PPC but only for the first cycle of wetting and drying. A further increase in number of cycles results in the OPC stabilized soil losing its strength whereas PPC stabilized soil keeps on gaining strength. A possible reason for the loss in strength of the 7.5% OPC stabilized soil after the first cycle of wetting and drying may be due to the higher heat of hydration of OPC. After the first cycle of wetting and drying, the moisture supplied by the wetting process may have induced better hydration of the OPC. OPC hydrates faster and generates more heat of hydration compared to PPC. This may have led to shrinkage cracks during the drying cycle resulting in a compromised microstructure of the OPC stabilized soil. Cuisinier et al. [31] state that some investigations have revealed that imposition of the first cycle of wetting and drying could induce a significant change in the microstructure of the soil. As a result, the strength of OPC stabilized soil decreases after the first cycle of wetting and drying. Introduction of pozzolanic materials like flyash in blended cements reduces the rate of hydration when compared to OPC [29], [30]. Thus, PPC hydrates slower than OPC and has comparatively lesser heat of hydration.This may have resulted in the steady increase in the strength of PPC stabilized soil with increase in number of wetting and drying cycles. However, microstructural investigations need to be carried out in future investigations to confirm the veracity of this possible mechanism. The gain in strength of PPC treated soil stabilizes for the third cycle of wetting and drying. But the strength of PPC stabilized soil after three cycles of wetting and drying is clearly higher than OPC stabilized soil. Thus, it can be stated that PPC stabilization of soil will perform better than OPC stabilization of soil under conditions of alternate wetting and drying, provided that the PPC content adopted for stabilization is sufficient enough. However, the determination of sufficient content of PPC in stabilization still needs to be evaluated as this study adopted only trial contents of PPC.

To get a better understanding of the performance of the OPC and PPC stabilized soil under conditions of wetting and drying, the strength index (I_{qu}) of the stabilized specimens were calculated for different cycles of wetting and drying. Muntohar and Khasanah [26] report strength index to be the ratio of strength of the stabilized specimen subjected to wetting-drying cycles to that of the strength of the specimens not exposed to wetting and drying. Figure 5 shows the strength index of the OPC and PPC stabilized specimens with number of wetting and drying cycles. The strength index of 7.5% PPC stabilized soil steadily increases from 1 to 2.63 for three cycles of wetting and drying. Even at lower binder content of 2.5%, the strength index increases from 1 to 1.61 for three cycles of wetting and drying, despite a marginal drop in index after the first cycle of wetting and drying at 0.95. On the other hand, the strength index of OPC stabilized soil only improves in the case of 2.5% OPC content. It increases from 1 to 1.17 for three cycles of wetting and drying. At higher content of 7.5%, the strength index increases only after the first cycle to 1.41, whereas on further increase in the number of cycles, the index drops and stabilizes to around 0.96. Thus, it can be stated that irrespective of binder content, PPC stabilized specimens perform better under conditions of wetting and drying compared to OPC stabilized soil. However, this statement has to be considered along with the absolute strength developed by the two different binders. The absolute strength of OPC stabilized soil is significantly higher even after the first cycle of exposure. However, after subsequent cycles of exposure, even the absolute strength of OPC stabilized soil specimens is reduced below that of PPC stabilized soil. The absolute strength of PPC stabilized soil increased with the increase in exposure cycles. However, this is true only for the higher binder content of 7.5%. In the case of low binder content of 2.5%, the absolute strength of PPC is less than OPC stabilized soil, though it steadily increases with the increase in number of cycles of wetting and drying.

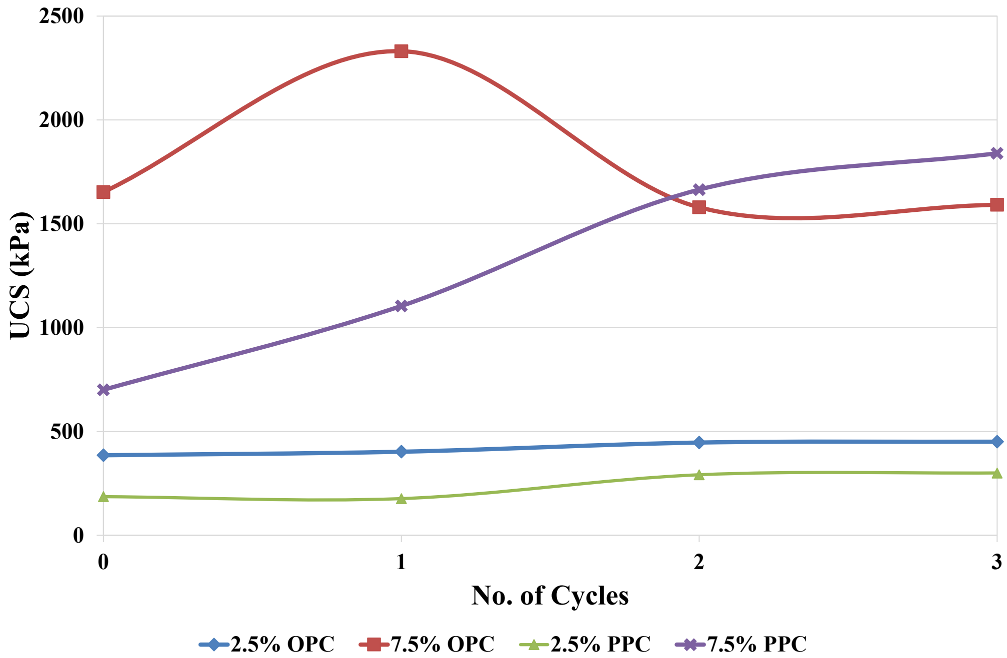

Figure 4. Strength of OPC and PPC stabilized soil subjected to wetting and drying cycles

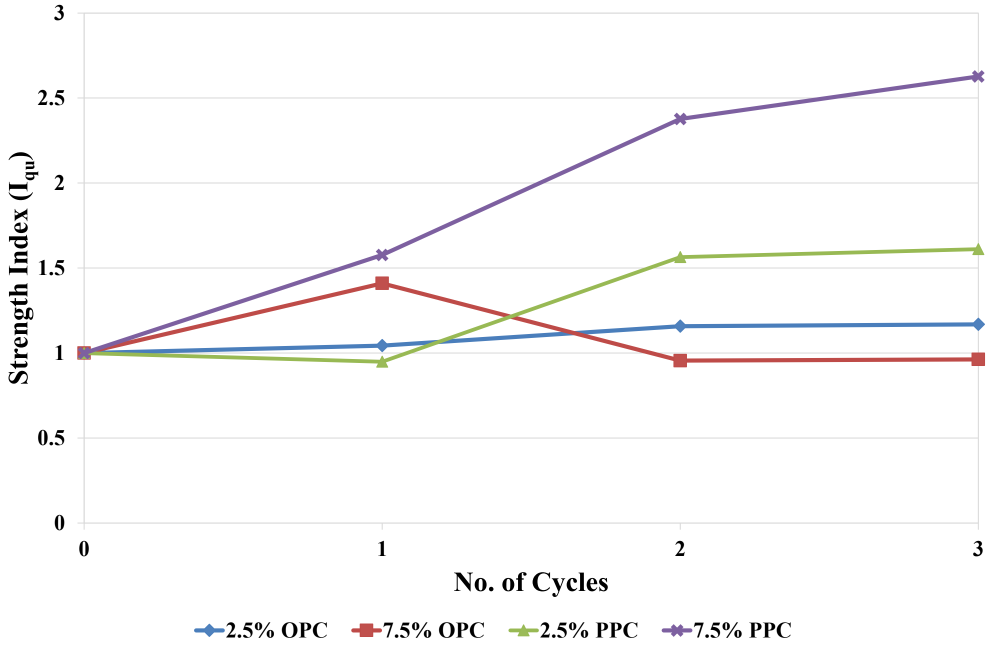

Figure 5. Strength index of OPC and PPC stabilized soil with wetting-drying cycles

4.3 Stress-strain characteristics of OPC and PPC stabilized soil

Figure 6 shows the stress-strain characteristics of the stabilized soil with 2.5% binder content. From the figure, it can be seen that 2.5% OPC stabilized soil behaves like a brittle material with the failure strain at 1.3%.

The first cycle of wetting and drying results in an increase in the failure strain to 4.5%. However, on further increase in wetting and drying cycles, the failure strains decrease to 2.9% and 1.6% respectively for 2 and 3 cycles of wetting and drying. Thus, in the case of OPC stabilized soil, it can be seen that the first cycle of wetting and drying has a major impact on the stiffness of the material. Subsequent cycles result in the material reverting to brittle behaviour.

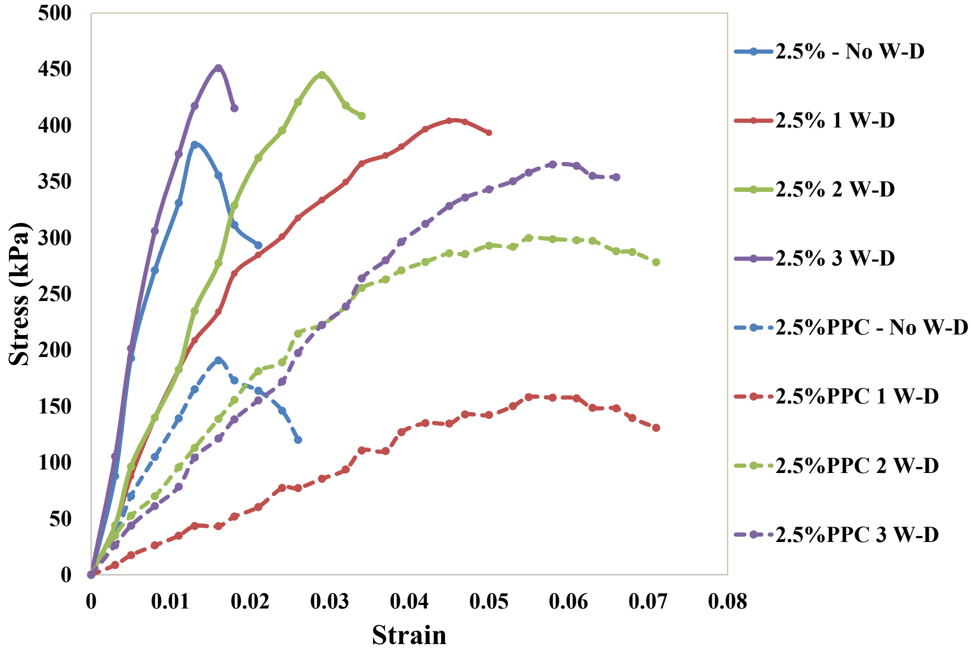

Figure 6. Stress-strain characteristics of 2.5% OPC and PPC stabilized soil

On the other hand, in the case of 2.5% PPC stabilized soil, the failure strain is 1.6%, indicating brittle behaviour. Similar to OPC stabilized soil, the first cycle of wetting and drying results in the material behaving more like a ductile material with failure strain as high as 5.5%. However, the effect of subsequent wetting and drying cycles is insignificant as PPC stabilized soil continues to behave as ductile material for all cycles with failure strains close to 5.5% for all cycles. Muntohar and Khasanah [26] report that the variation in moisture content of stabilized specimens is maximum during the first cycle of wetting and drying. This additional moisture content supplied during the first cycle of wetting and drying may have also resulted in further formation of cementitious products leading to a gain in strength after the first cycle. A similar response has been reported in literature by earlier investigators [26], [28], [32]. Thus, the resultant effect of increased moisture content and further formation of cementitious products may be a reason for the ductile behaviour of the stabilized specimens after the first cycle of wetting and drying in the present study. Thus, it can be concluded that the first cycle of wetting and drying reduces the stiffness of the stabilized specimen, irrespective of binder type. Multiple cycles of wetting and drying influences the ductility behaviour of OPC stabilized specimen whereas its effect on the ductility behaviour of PPC stabilized soil is insignificant at low binder content of 2.5%.

Figure 7 shows the stress-strain characteristics of stabilized soil with 7.5% binder content. At higher binder content of 7.5%, OPC stabilized soil exhibits more brittle behaviour compared to PPC stabilized soil. However, in the case of 7.5% OPC stabilized soil, the soil exhibits increased ductile behaviour until 2 cycles of wetting and drying, which was not the case in 2.5% OPC stabilized soil. On further increase in wetting and drying cycles to 3, the soil started to exhibit brittle behaviour. The failure strains increased from 1.8% to 5.5% for 2 cycles of wetting and drying and then again reduced to 2.6%. In the case of 7.5% PPC stabilized soil, the failure strain increases from 2.9% to 5.8% for 2 cycles of wetting and drying and then reduces to 2.9% for the third cycle. Thus, it can be seen that at higher binder content of 7.5%, wetting and drying cycles render both OPC and PPC stabilized soil to behave like a more ductile material until two cycles of wetting and drying beyond which both tend to exhibit brittle behaviour once again. At higher binder content, the formation of cementitious gels would be more pronounced. This would have delayed the ingress of moisture, thereby rendering both the materials to behave in a ductile manner until two cycles of wetting and drying. Further increase in number of cycles, the formation of more cementitious products would have overshadowed the softening effect due to wetting and drying cycles resulting in exhibition of brittle behaviour. Aldaood et al. [33] report that the increase in number of wetting and drying cycles results in the formation of macropores. The loss in strength of OPC stabilized soil at higher number of wetting and drying cycles may also be due to the formation of such macropores. It can be clearly seen that the stress-strain behaviour of the stabilized specimen is influenced by binder content, binder type and cycles of wetting and drying. Overall, it can be stated that PPC performs better than OPC at higher binder content and higher number of wetting and drying cycles. However, the number of cycles considered in this investigation is too low for giving a generalized verdict on the performance of PPC in comparison with OPC in soil stabilization.

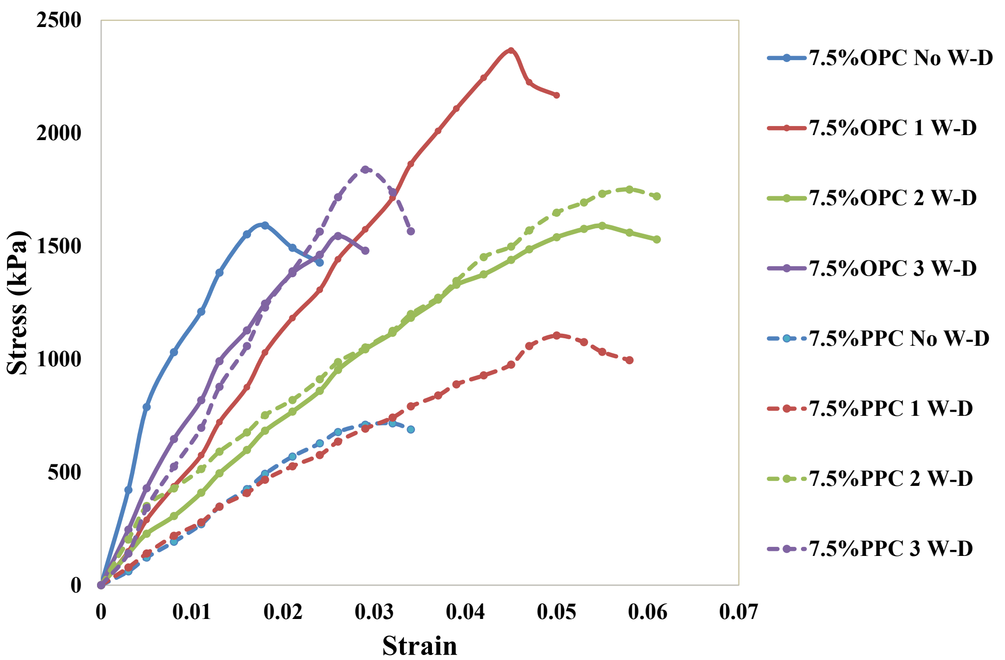

Figure 7. Stress-strain characteristics of 7.5% OPC and PPC stabilized soil

The behaviour of the stabilized soil can be further interpreted from the stress-strain characteristics using the secant modulus of elasticity [26]. The secant modulus of elasticity, E_{50} is defined as the ratio of half of peak stress to the corresponding axial strain [34]. Figure 8 shows the variation of secant modulus, E_{50}, for the various combinations of binders used in this investigation for different cycles of wetting and drying. It is evident from the figure that 7.5% OPC stabilized soil has the higher modulus when not subjected to any wetting and drying cycles. Increase in the number of cycles results in the modulus of OPC stabilized soil to reduce. This is true for both 2.5% and 7.5% binder content. However, after two cycles of wetting and drying there is an increase in the secant modulus of OPC stabilized soil for both 2.5% and 7.5% binder content. The secant modulus drops from 207.1 MPa and 36.53 MPa to 34.24 MPa and 18.13 MPa after two cycles for 7.5% and 2.5% OPC stabilized soil, respectively. On the other hand, the secant modulus of 2.5% PPC stabilized soil does not show a clear trend. It starts at around 10.3 MPa and wavers up and down to 5.74 MPa for three cycles of wetting and drying. However, 7.5% PPC stabilized soil, clearly shows an increase in secant modulus with increase in wetting and drying cycles. The modulus of 7.5% PPC stabilized soil increases from 24.33 MPa to 67.18 MPa, in comparison with the 76.77 MPa achieved by 7.5% OPC stabilized soil after three cycles of wetting and drying.

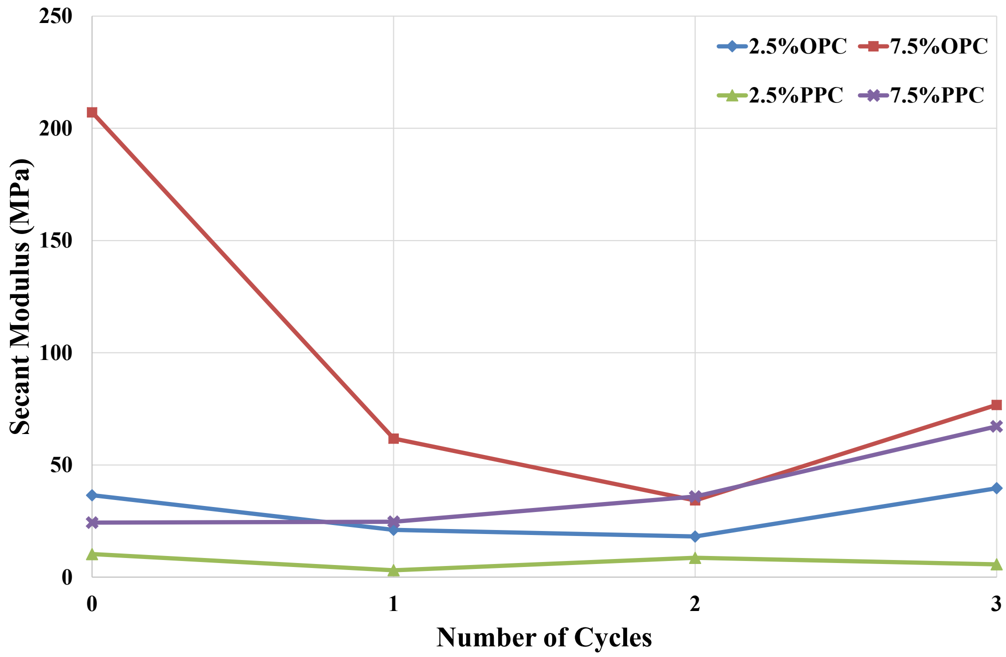

Figure 8. Secant modulus (E_{50}) of OPC and PPC stabilized soil

5 Conclusions

The present investigation aimed to study the efficacy of PPC in stabilization of soil under conditions of alternate cycles of wetting and drying. From the results of the investigation, the following points may be concluded.

i. The strength of OPC stabilized soil was much higher than that of PPC stabilized soil for both stabilizer contents investigated. In fact, the strengths developed by OPC was more than twice that of PPC for both the stabilizer contents. Thus, it can be concluded that under normal conditions of stabilization, OPC stabilization performs much better than PPC stabilization, irrespective of binder content.

ii. The durability performance of both OPC and PPC stabilized soils at low binder content of 2.5% resulted in a marginal increase in strength for both. However, at higher binder content of 7.5%, the durability performance of both OPC and PPC stabilized soil were different with strength loss for OPC stabilized soil against strength gain for PPC stabilized soil. Thus, it can be stated that the durability performance of the stabilized soil is influenced by binder type as well as its content.

iii. Under conditions of wetting and drying, the strength index of PPC stabilized soil increased with the increase in number of cycles of wetting and drying for both binder contents. For OPC stabilized soil, the strength index increased with the increase in number of cycles at low binder content and decreased with increase in number of cycles at higher binder content. The values of increase in strength index were also higher for PPC stabilized soil when compared to OPC stabilized soil. Thus, it can be postulated that PPC stabilization of expansive soil performs better under conditions of alternate wetting and drying when compared to OPC stabilization.

iv. The first cycle of wetting and drying results in the increase in failure strains of both OPC as well as PPC stabilized soils. Thus, it can be stated that irrespective of binder type or content, wetting and drying cycles can change the straining behaviour of the material.

v. At low binder content of 2.5%, with increase in wetting and drying cycles, OPC stabilized soil recovered back its brittle behaviour whereas up to three cycles of wetting and drying did not have any influence on the ductile behaviour of PPC stabilized soil. Thus, use of PPC stabilization at low binder content may not provide satisfactory performance.

vi. At higher binder content of 7.5%, the failure strains increase with the increase in wetting and drying cycles up to 2 cycles after which both OPC and PPC stabilized soils exhibit brittle behaviour but the PPC stabilized soil retains higher strength when compared to OPC stabilized soil, thus reinforcing the conclusion that PPC stabilized soil performs much better than OPC stabilized soil provided that sufficient binder content is available for satisfactory performance.

The number of cycles considered in this investigation, however, can only give initial indications and higher number of cycles of wetting and drying is essential for getting a clearer picture for more obvious conclusions as to its performance under wetting and drying conditions.

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Conflict of interest

The authors declare no conflict of interest in the publication of this manuscript.

Data availability

The data obtained during the course of this investigation is available on request from the authors.

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Preliminary report

Compressive strength of green concrete with low cement and high filler content

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ABSTRACT

It is estimated that by the end of the current decade, about 26% of clinker will be replaced by suitable alternative materials, among which limestone filler probably has the greatest potential due to its wide availability and relatively low price. Although codes allow as much as 35% of clinker to be replaced by limestone filler, the amounts used in practice are far lower and average only 6-7% globally. In order to use the great potential of fillers, it is necessary to know the key parameters that affect the properties of green concrete with low cement and high filler content in fresh and hardened states. Therefore, thorough literature review was conducted, followed by design of 18 concrete mixtures, in order to analyze the effects of the percentage of cement replacement, water-cement ratio, but also the particle size of the applied filler. The results of experimental research show that at the same w/c ratio there is an increase in compressive strength with increasing percentage of cement replacement and that it is possible to make medium-strength concrete with significantly reduced amount of cement 180 kg/m^3 compared to traditional concrete. Thus, economic benefits can be achieved, but also the negative impact of the concrete industry on the environment can be reduced. Unfortunately, the workability of such mixtures can be impaired to the extent that their practical application is prevented, so it is necessary to take further steps to solve this problem.

1. Introduction

The human population lays claim to use all natural resources and healthy environment; but it also should not ignore the moral obligation - not to deny those same rights to some future generations. The use of fossil fuels (including cement production) is the main problem for creating the greenhouse effect and climate change. Moreover, it is estimated that about 7-8% of total CO_2 emissions of anthropogenic origin is a consequence of cement production, with a further tendency to grow, despite the constant improvement of technology and increasing efficiency [1,2].

In addition, the research of some authors [3–5] indicates that by the end of the current decade the world production of cement will exceed five billion tons, which, unlike in 1990, when one billion was produced, represents an increase of as much as five times. The direct consequence of this activity is an increase in global CO_2 emissions for additional 1-2% [6]. Having this in mind, the cement industry is facing numerous challenges including increasing pressures from public. The biggest concern is the use of a large amount of relatively expensive energy, which participates in the total production costs with 30-50% [5], and the final output is a product with significant CO_2 emission. Accordingly, the concrete industry can play one of the key roles on the path of sustainable development and rational use of resources.

In order to preserve the competitiveness of concrete as the most used construction material in the future, it is necessary to take appropriate measures to promote sustainable development and environmental protection in this field. One has to take into account the huge amount of cement produced, as well as the fact that the production of one ton of pure Portland cement emits about 860 kg of CO_2 into the atmosphere [7–9]. Therefore, it is concluded that even a small replacement of a certain amount of cement with alternative materials can have a significant global contribution in solving these problems. About 60% of the total share of CO_2 emissions, is the result of the decomposition of limestone during the decalcification process, while the remaining 40% is the result of burning fossil fuels to obtain the required amount of energy in the production process [10,11].

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In 2017, the average clinker content in cement was around 77%, and by the end of the decade, a reduction of an additional 3% is planned, which means that 26% of clinker will be replaced by suitable alternative materials [11]. These materials can be artificial (fly ash, blast furnace slag) or natural (limestone, quartz and other). The previous attention of researchers was mainly focused on the application of industrial by-products [12–14], primarily because of their appropriate pozzolanic properties. However, the latest research in the world [15–17] also show a very large potential of inert and weakly reactive materials, which are mainly different types of fine powder materials (fillers).

Potential problems and major limitation in the wider application of alternative binders are the limited quantities of waste materials based on industrial by-products that are in constant decline [18] and the relatively high cost of other supplementary cementitious materials. On the other hand, the potential of fillers is further enhanced by relatively low unit price, wide availability (Figure 1), and simple production processes with minimal CO_2 emissions of only 8 kg/t [19], so we should expect higher demand for these materials in the future.

As the largest cement producer in Serbia, the company Lafarge Srbija Ltd. managed to reduce the amount of CO_2 emitted per ton of cement from 1990 to 2014 by 21%, and by 2030 a reduction of an additional 19% is planned [20].

2 A brief overview of historical development and regulations

There is no universal definition of filler [15]. It can be said that they are very fine powdery materials that are mostly inert or weakly reactive [15,21], and can be of different origin and mineralogical composition. Limestone fillers are the most common and most widely used [18], but equally satisfactory results can be found in the literature when using quartz fillers [22]. However, the higher price of quartz fillers should be also considered, which is mainly due to the more complex technological process of production, less availability, but also their serious harm (potentially cancerogenic) to human health should be taken into account [18,23].

The first use of fillers as a replacement for cement was recorded at the beginning of the last century [15]. During the construction of Arrowrock and Elephant Butte Dams (Figure 2), in order to reduce the internal temperature due to excessive heat of hydration and shrinkage of these massive concrete structures, almost 50% of the cement was replaced by ground granite and sand extracted during construction [15,24,25].

These impressive engineering endeavours were followed by the first significant long-term tests of the mechanical properties and durability of concrete in which cement was replaced by fine powder materials [26]. However, the more serious application of fillers in the cement industry has been waiting for almost fifty years [3]. Due to the consequences of the great oil crisis in 1973, the somewhat larger use of fillers as a substitute for clinker began. A period of standardization [15] followed, as well as the legally regulated application of fillers in the cement industry. In the beginning, the regulations were quite conservative and these were very modest figures (up to 5%), but over time the regulations were innovated and allowed an increasing percentage replacement of clinker by up to 35% [27,28]. However, it was common to find standards that were probably non-conservative at the time of publication, because of all the uncertainties related to the technology of filler production, Figure 3. Thus, in the Spanish standard from 1960, 10% of fillers were allowed, and already in 1975 it was increased to 35%, which is still an upper limit for most countries.

A review of world standards found that in 1991, about 25 countries around the world allowed only 5% of cement fillers[28], while in 1993 about 40 countries around the world adopted a limit of 20% [27].

Today, many modern regulations in this area limit the maximum percentage of fillers (exclusively of limestone origin) in cement. It can be concluded that the regulations of some of the most developed countries still have rather conservative restrictions, maximum of 15%, and these countries are: China (2007), Canada (2008) and USA (2012) [15].

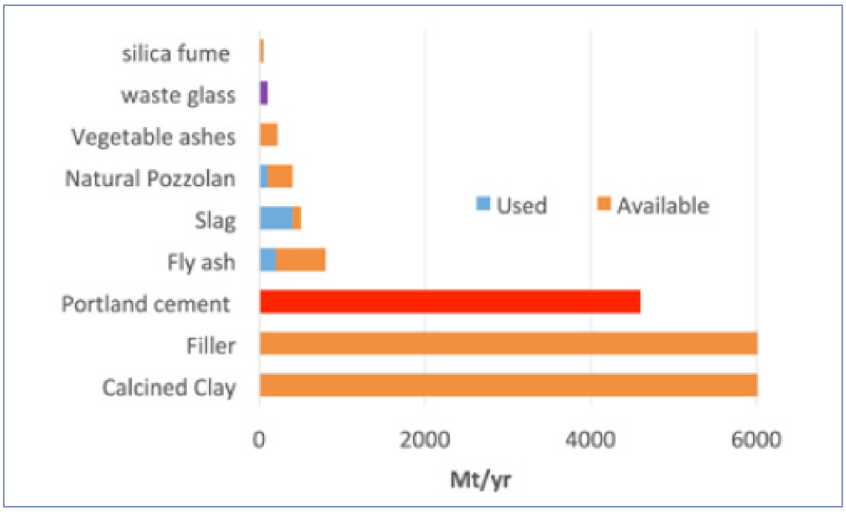

Figure 1. Estimated availability and current use of certain alternative cementitious materials in the world [3]

Figure 2. Arrowrock Dam on Idaho’s Boise River (left) and Elephant Butte Dam on the Rio Grande (right) [24,25]

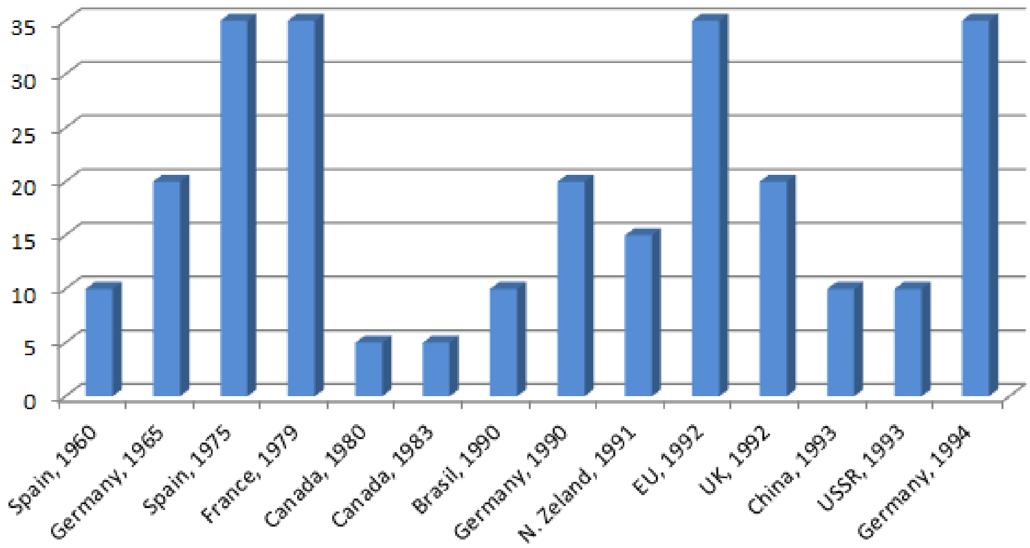

Figure 3. The maximum percentage content of limestone filler in cement and year of publication of the standard over the period 1960-1995.

Unfortunately, according to World Business Council for Sustainable Development (WBCSD)[29], the quantities of fillers that are applied in practice are far below the maximum allowed values from the regulations. Average limestone filler content in cement according to WBCSD [29] in the countries of the European Union, as well as in the whole world, by years is shown in Figure 4.

A similar trend is observed in Europe and in the world. Since the beginning of the 1990s, there has been a very slight increase to some 6-7% and from 2011 until today it remains practically constant. The same applies when it comes to other alternative materials (Figure 5), for the countries of the European Union. It is obvious that limestone filler is the most represented mineral addition in cement together with slag, but its content is even five times lower than the allowed values [30].

As a positive example, Morocco, Algeria, and Tunisia should be pointed out; in these countries, the average content of limestone filler in cement is the highest in the world and from 2011 until today it is about 20-22%. [29]. In addition, in these countries, the limestone filler is twice as common as all other mineral additives combined.

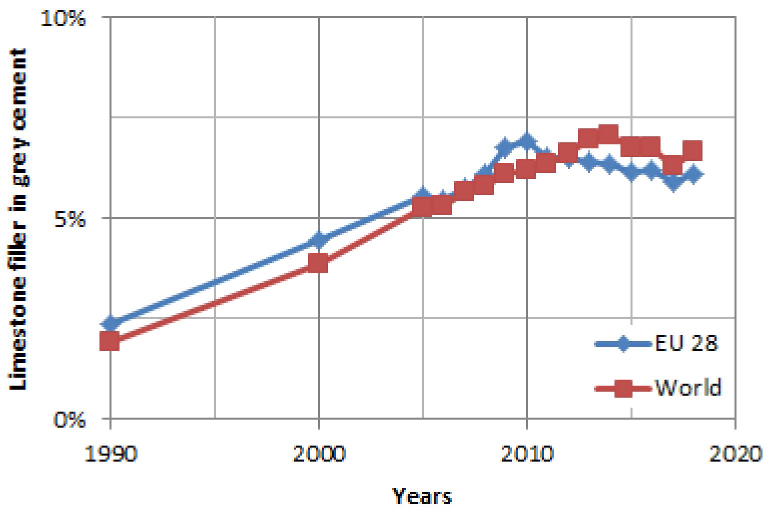

Figure 4. Limestone filler content in cement (chronological development), data from WBCSD [29]

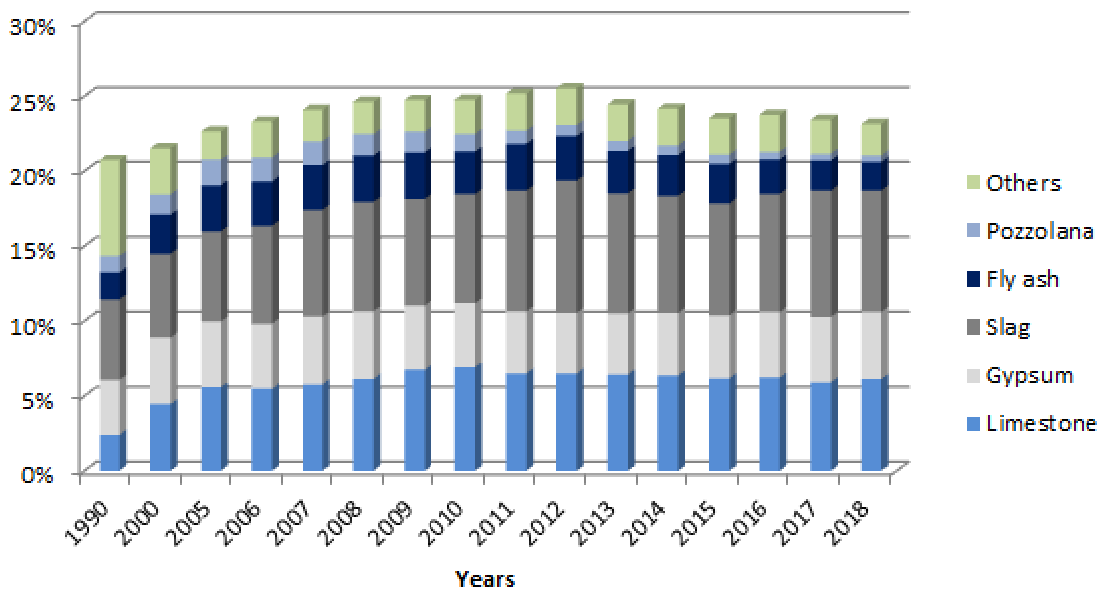

Figure 5. The percentage content of individual mineral additives in cement (chronological development), EU countries, data from WBCSD [29]

Here it should be noted that in addition to the maximum allowed content of limestone filler in cement, modern regulations introduce some additional restrictions. Thus, the EN 197-1 [30] defines the minimum mass content of calcium carbonate ($CaCO_3$) which must not be less than 75% of total filler content, while the clay content must not be more than 1.2 g per 100 g of filler. In the case of limestone used in cements, there are two designations, L and LL in dependence of the Total organic carbon content (TOC). In the case of L-limestone, this content must not exceed 0.5% of the weight of limestone, while in the case of LL-limestone this condition is stricter, and must not exceed 0.2%. In the EN 12620 [31] certain grading requirements are defined for filler aggregate, table 1.

Table 1. Grading requirements for filler aggregate, EN 12620 [31]

Particle size [mm]	Passing by mass [%]
2	100
0.125	85-100
0.063	70-100

3 Influence of fillers on the properties of fresh and hardened concrete - a literature review

Experimental tests on the filler application have gained substantial importance after technology modernization and technological production process improvement, which is especially evident in the last 10-15 years. The influence of the filler on the properties of concrete in a fresh and hardened state can be crucial for practical application, so it is necessary to study it in detail, in order to take advantage of all the positive effects, and hopefully reduce negative ones.

3.1 Workability of fresh concrete

Divided opinions about the influence of fillers on the workability of concrete mixtures can be found mainly in older papers [27,28], some authors reporting a positive [32], and others a negative experience [33]. Analysing the replacement of up to 40% of cement with fillers, the authors [33] conclude that in the case of water-cement (w/c) ratio was constant, workability slightly decreases when the filler content increases, due to the increase of specific surface of aggregate, and vice-versa: when the water-powder (w/p) ratio is constant, workability improves with increased filler content. Because of that, the same amount of water is adopted in all mixtures, i.e. the water-powder ratio was constant, w/p=0.5. On the other hand, replacing up to 20% of cement, in [34], there was no change in the workability of concrete mixtures (for the same w/c), regardless of the filler content. In recent research [32] positive effects (increase of limestone resulted in decrease of superplasticizer demand for the same flow value) are dominantly recorded. This is probably a consequence of the mentioned modernization and improvement of technological process of filler production, primarily the improvement of the grinding process. In addition, the application of new generation chemical additives - high-performance superplasticizers provides high water reduction. Obtaining a proper range of very fine particles is primarily reflected in the increase in packing density, which leads to the reduction of voids to be filled with water [15]. This allows additional reduction of water. However, it is practically always necessary to apply a certain amount of superplasticizer, in order to preserve the workability of the mixture [32,35]. Rezvani et al. [32] deals with workability of concrete, in which as much as 65% of cement is replaced by filler. The authors state that the reduction of water below a certain limit has a huge impact on workability. However, all mixtures with water-cement (w/c) ratio greater than 0.25 had satisfactory workability (plastic viscosity was less than 50 Nm/(m/s)). In addition, the authors determined the optimal amount of paste (water, cement, and filler) and it was 320 kg/m^3 in all mixtures, while the superplasticizer is dosed as needed, until the mentioned workability of the mixture is achieved. A similar procedure was applied in [36,37], where the amount of paste was 20-34% of the total volume of concrete, while the superplasticizer was dosed in the amount of 0.5-2% by the weight of powder component, in order to provide the desired slump of 20 cm.

In the same way, Proske et al. [38] found that the content of fine particles (<0.125 mm) should be at least 430 kg per 1 m^3 of concrete (kg/m^3) for maintaining satisfactory workability. In addition, it was found that the amount of water can be reduced (which is necessary if the goal is to obtain high compressive strength) to a minimum of 125 kg/m^3. Therefore, workability of the mixture must be corrected by adding high-performance superplasticizers (2-6.5 kg/m^3). Additionally, the same authors, in order to obtain better workability of the mixture, suggest the use of fillers of higher fineness, which is in agreement with the other research [39].

3.2 Influence of filler on the compressive strength

3.2.1 Cement replacement percentage and particle size distribution

The compressive strength of concrete is probably the most important parameter in the overall assessment of concrete quality and practically all physical and mechanical properties can be expressed in the function of this property.

Analysis of the replacement up to 40% of cement with limestone filler found that the content of filler up to 10% has insignificant effect on the decrease in strength (moreover, slight increase was observed), while for higher replacement percentages there is a gradual reduction in compressive strength [33]. According to the author, this is a consequence of intergranular voids being filled by the filler and additional portion of filler begins to take place of the main aggregate grain. Although the term water-binder (w/b) ratio is mostly used, which was constant in all mixtures and it was 0.5, which is actually the w/p ratio. Other papers [34,40,41] also use the approach of adopting w/b = const, and as it is about inert or weakly reactive materials, a certain amount of excess water that cannot hydrate will appear, which results in an increase of the real w/c ratio, and wrong conclusions can be drawn, Figure 6. Therefore, one of the important conclusions is that when replacing cement with filler, the w/c ratio should be kept constant or reduced in the case of a large replacement, in order to avoid a reduction in compressive strength [35,38].

Volumetric replacement of cement paste (cement + water) with limestone filler of similar particle size was performed by Chen et al. [35,36]. In these studies, the filler volume was given as a percentage of the total concrete volume and it was 0-12%, while the w/c ratio was in the range of 0.35-0.6. At the same w/c ratio, an increase in strength with an increase in filler content was recorded, and it was higher in samples with a higher percent of replacement and with higher w/c ratios reaching 25-30% in some cases. Workability was provided by the use of plasticizers (1-2% by the weight of the powder component), and the slump was over 20 cm. Researchers from Germany came to the similar conclusions [17,38,42–45], when replacing a large amount of cement (up to 50%). For the same w/c ratio, samples containing fillers had higher strengths than the reference ones. The difference is greater in samples with higher filler content. High-performance superplasticizers (2-7 kg/m^3) were used to obtain the desired workability of the mixture (flow table diameter 490-550 mm; plastic viscosity 50 Nm/(m/s)).

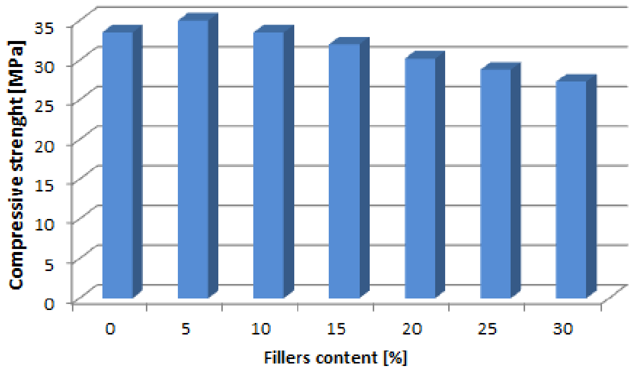

Figure 6. Compressive strength of the samples as a function of % cement replacement, samples measuring 10x10x10 cm, data from Bederina et al. [33]

John et al. [15] states that it is possible to replace as much as 75% of the binder (cement + other binders) with fillers and for the amount of binder of only 100 kg/m^3, extremely high strengths, up to 90 MPa, can be obtained. However, this is a serious problem, because, in order to achieve this, a large reduction of water and application of new generation superplasticizers are necessary. Even then, it is difficult to achieve and control acceptable workability of the mixture. The authors of this paper did not establish a possible relationship between the compressive strength and the w/p ratio, but a certain relationship between the compressive strength and the w/c ratio was established.

The fineness, i.e. the particle size distribution curve of the filler also has a significant effect on the compressive strength [21,38,41,43]. Practically all studies have found an increase in strength about 10-20% when using fillers of higher fineness. This is more pronounced in samples with higher percentages of replacement and in some cases was as much as 50% [38,43]. This phenomenon is probably a consequence of the higher particle packing optimization.

3.2.2 Mineral composition of fillers

The study by Muller et al. [46] is one of the most important researches in which quartz fillers were used. In order to increase the range of fine particles, the replacement of cement (4-10% of the total volume) was performed using two fillers of different particle sizes. Compaction – Interaction Packing Modelproposed by Fennis [47] for increasing the particle packing was applied, and types of cement of relatively large strength classes, CEM I 52.5R (68MPa) and specially produced cement of increased fineness of grinding (micro cement 106.3 MPa) were used. In some mixtures, the amount of cement was only about 110 kg/m^3. The obtained concrete compressive strengths ranged from 60 MPa, up to 102 MPa (CEM I 268 kg/m^3). Polycarboxylate-based superplasticizers (5-6 kg/m^3) were used, w/c ratios were in the range of 0.4-0.7, but nevertheless workability of mixtures in which the amount of cement was less than 138 kg/m^3 was not reported.

The influence of fillers of different origin, as well as the fineness of the mill was analysed in [22]. Quartz fillers (finer and coarser than cement) were used in this study, as well as syenite and wollastonite fillers, which are also finer than cement, and up to 50% of the cement was replaced. The reference samples had the lowest strength, and the increase in filler fineness was accompanied by an increase in strength, for practically all w/c ratios, regardless of mineral composition.

Similar conclusions have been drawn by other researchers [40]. When replacing 50% of cement with fillers of different mineral compositions, and a constant w/c ratio of 0.5, an increase in compressive strength of as much as 50 to 144% was observed in relation to the reference concrete mixture, Figure 7 (left).

The influence of the density of the filler on the initial volumetric concentration of water is very important and it is reflected in the porosity of the hardened paste [40], Fig. 7 (right). Fillers with lower density will result in a higher content of powder component, and as it seems, higher strength, Figure 7 (right).

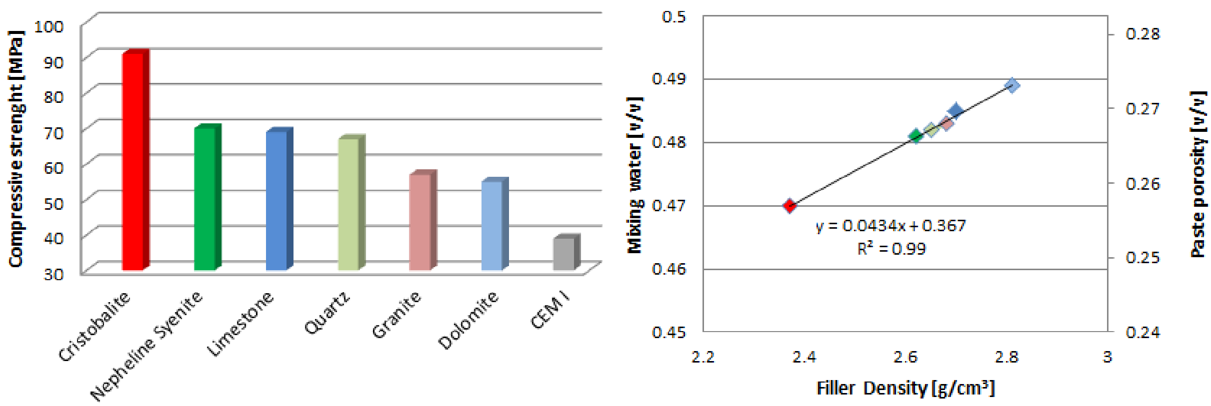

Figure 7. Influence of mineral composition of filler on compressive strength (left), required amount of water and porosity (right), data from Damineli [40]

4 Objectives and methodology

The literature review shows a clear trend towards improved efficiency in cement production, primarily the development of new and cleaner technologies that may require significant investment. One way to achieve these goals may be simply replacing a certain amount of cement with a cheaper and cleaner product – filler.

In this study, different green concrete mixtures with low cement and high filler content were made for structural application. Green, i.e. eco-friendly, concrete has a reduced negative environmental impact compared to conventional concrete. Notwithstanding the lower binder content, the concrete performance had to be conserved.

Workability must be satisfactory for practical application, so the slump-test value according to SRPS EN 12350-2 [48] was chosen to be more than 15 cm (class S4), typical for pumping concrete. The mean compressive strength at 28 days had to be in the range 38-45 MPa, in order to obtain compressive strength class C25/30 or C30/35, providing the structural applications of these concretes.

An extensive test program was conducted including testing physical and mechanical properties of the fresh and hardened concrete. One of the aims of this study was to show that different fillers can be efficiently used to reduce cement amount and concequently negative environmental impact. In addition, the influence of different percentage of cement replacement with filler, as well as the amount of water in the mixture (through w/c ratio) was observed. In addition, the influence of filler particle size was analysed on a limited number of mixtures.

5 Experimental program

5.1 Materials

In order to eliminate the influence of other mineral additives, pure Portland cement (max 5% additives) of the middle class (CEM I 42.5R) with accelerated strength gain was used for experimental tests. Two limestone fillers (L), different particle sizes, and one quartz (Q) filler were used as cement replacements. Particle size distribution of the cement and filler are shown in Figure 8. The figure shows that the filler in the L1 mark has much finer particles than cement, while the other two fillers L2 and Q have much larger particles than cement and fail to meet the requirements in terms of grading requirements from EN 12620 [31], Table 1. The solid green line in the Figure shows these constraints, while the dashed green line represents a linear extrapolation. Thus, the analysis partially covers the impact of filler grading size, especially those that go beyond the recommendations of the standard.

The chemical composition of the applied limestone fillers is in accordance with the standard EN 197-1 [30] ($CaCO_3$ content is 98%, $MgCO_3$ is 1.4%, Fe_2CO_3 is 0.1% and HCl insoluble content is 0.5%). Unfortunately, the chemical composition of the quartz filler was not provided by the manufacturer.

In order to improve the workability of concrete mixtures, a second-generation superplasticizer (based on polycarboxylate) was used. The amount of superplasticizer is shown in relation to the mass of the powder component (cement + filler), Table 2.

A three - fraction river aggregate (1850 kg/m^3) was used, with nominal maximum aggregate size of 16 mm, as well as water from the city water supply.

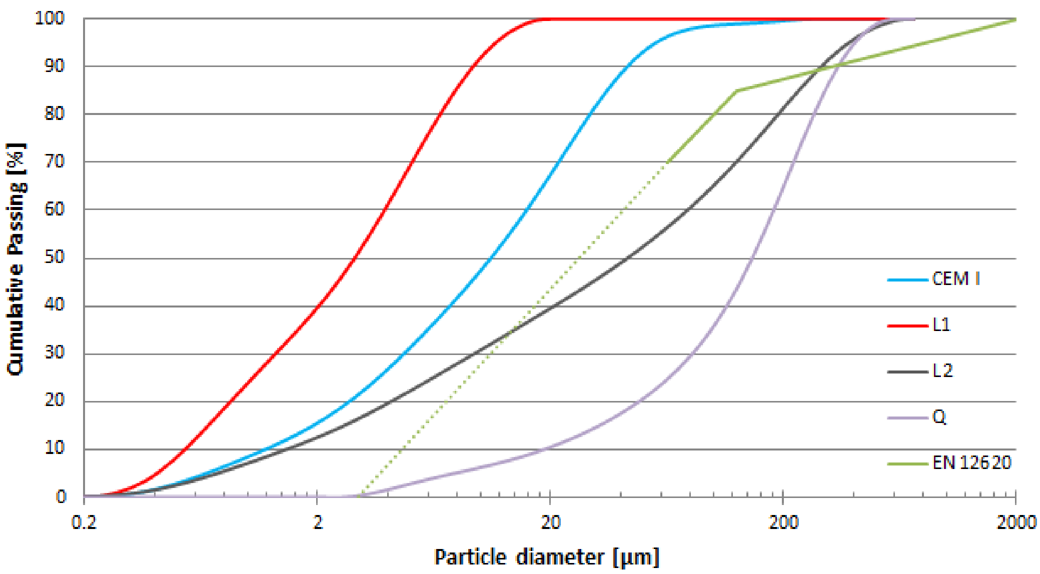

Figure 8. Particle size distributions of materials used

5.2 Selection of the optimal particle size distribution of the aggregate

One of the initial and most important tasks is the correct choice of the particle size distribution of the aggregate. The individual fractions in the mixture are combined so that the total particle size distribution curve is as close as possible to certain reference curves (ideal grading curves, optimization curves). This achieves good particle packing of the aggregate and thus reduces the remaining space that needs to be filled with fine particles, i.e. paste (cement, water, and filler), Figure 9.

Probably the most well-known continuous optimization curve was proposed by Fuller [22,49] and is given by equation (1):

$$P_{(d)} = \left(\frac{d}{d_{\max}} \right)^n \tag{1}$$

where:
$P_{(d)}$ - size cumulative distribution function
d – particle diameter being considered [mm]
d_{max} - the maximum particle size of the mix [mm]
n = 0,5 (0,45) - distribution modulus

This curve has been in use for over a hundred years and is still widely used today. The biggest disadvantage of the previous equation is that only the nominal maximum aggregate size is considered. Funk and Dinger propose modification on the basis of numerical simulations (for ideally spherical grains), equation (2), which also takes into account the size of the minimum particle size (d_{min}) in the mixture [22,47]. The value of the exponent that gives the highest packing density n = 0.37 is proposed.

$$P_{(d)} = \frac{d^n - d_{\min}^n}{d_{\max}^n - d_{\min}^n} \tag{2}$$

The final particle size distribution curves of the aggregate and filler mixtures for concrete (designations of the concrete mixutures will be explained in detail in part 5.3) are shown in Figure 10. The target is the Fank and Dinger curve, equation (2), and the exponent value of 0.37 is adopted. For comparison, the Fuller curve, equation (1), is also shown. Significantly higher content of finer particles of the adopted mixtures is clearly observed in relation to the Fuller proposal (n = 0.5), but also slightly higher in relation to the Fank and Dinger proposal. The content of these particles is higher for higher percentage of cement replacement, because in these mixtures there is a more pronounced filler effect. It should be borne in mind that, as previously mentioned, the Fuller curve, equation (1), does not take into account fine particles (particle size distribution curve of the aggregate), and when powder components are added to the mixture, this and the curve described by the equation (2) will consequently converge.

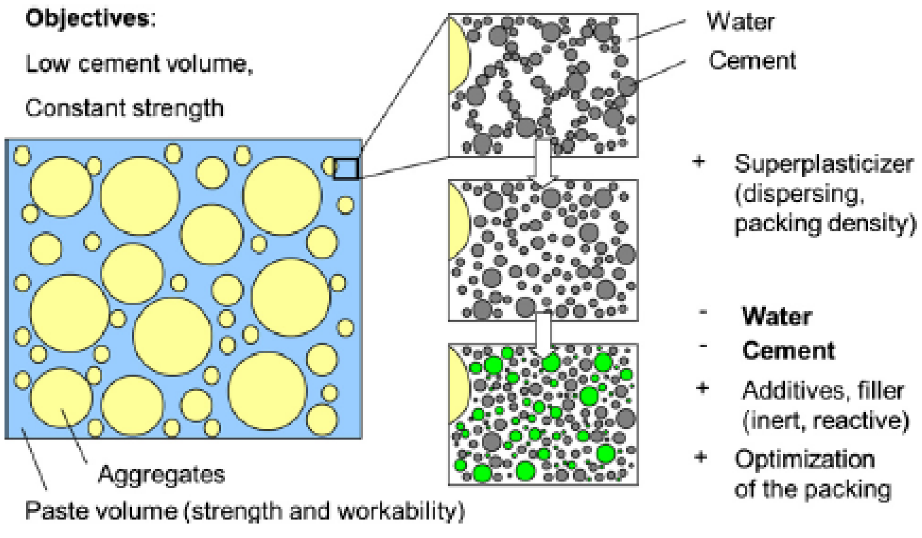

Figure 9. The principle of simultaneous reduction of cement and water with the addition of fillers [32]

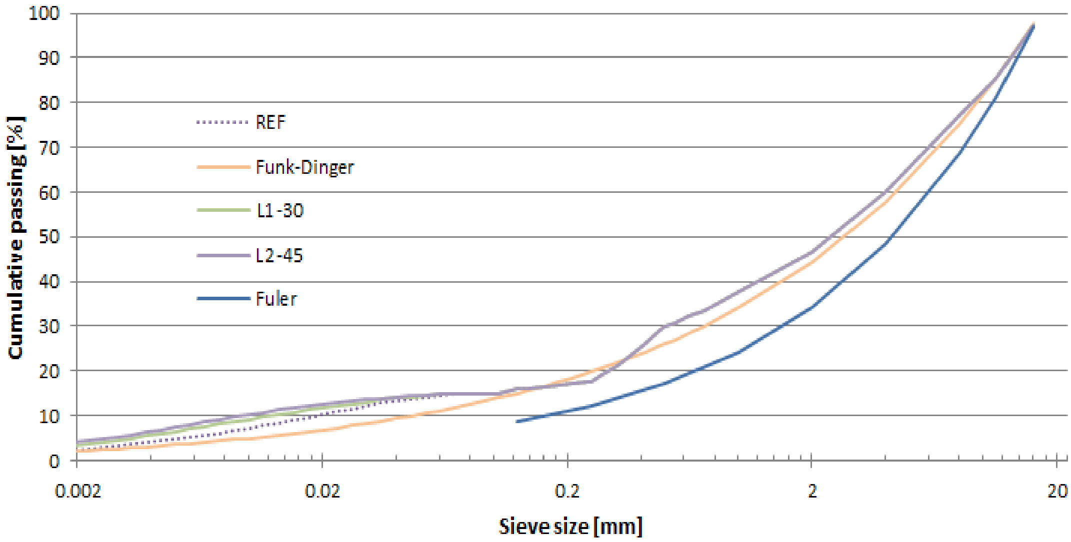

Figure 10. Particle size distribution of concrete mixtures

5.3 Concrete mix design

The composition of the designed concrete mixtures is shown in Table 2. Reference mixtures were made with 330 kg/m^3 of cement and w/c ratios of 0.54; 0.58; 0.62; 0.71. This is followed by mixtures in which 30% and 45% of cement by weight are replaced by filler. Primarily, the influence of L1 filler, which is significantly finer than cement (8 mixtures), was analyzed, and in order to include the influence of different filler sizes, 3 more mixtures per filler were designed using L2 and Q fillers.

Table 2. Mix design of concrete

Mix	Filler* d_{50} [μm]	CEM [kg/m^3]	Filler [kg/m^3]	Water [kg/m^3]	S.plast. %	w/c	w/p	Slump [cm]
Ref-1	/	330	0	177	0.5	0.54	0.54	23
Ref-2	/	330	0	190	0	0.58	0.58	15
Ref-3	/	330	0	204	0	0.62	0.62	17.5
Ref-4	/	330	0	233	0	0.71	0.71	23.3
30_L1_1	2.8	230	100	124	2	0.54	0.38	16.3**
30_L1_2	2.8	230	100	134	1	0.58	0.41	19.5**
30_L1_3	2.8	230	100	144	1	0.62	0.44	19.7
30_L1_4	2.8	230	100	163	1	0.71	0.49	21
30_L2_1	42.7	230	100	124	2	0.54	0.38	3.2
30_Q_1	146.8	230	100	134	2	0.58	0.41	0.5
45_L1_1	2.8	180	150	97	2	0.54	0.29	0
45_L1_2	2.8	180	150	105	2	0.58	0.32	9.3**
45_L1_3	2.8	180	150	112	2	0.62	0.34	17.7**
45_L1_4	2.8	180	150	127	2	0.71	0.38	19.7**
45_L2_1	42.7	180	150	97	2	0.54	0.29	0
45_L2_2	42.7	180	150	127	2	0.71	0.38	1
45_Q_1	146.8	180	150	112	2	0.62	0.34	0.5
45_Q_2	146.8	180	150	127	2	0.71	0.38	1

* mean particle size
** shear slump, Fig. 11

Figure 11. Possible forms of slump (true slump - left, shear slump - right)

Mixtures have the following designations: X_Y_Z, where X represents the percentage of cement replacement (30, 45), Y denotes the filler used (L1; L2; Q), while Z is the ordinal number of the mixture.

The mixing process lasted a total of 4 min, 1 min of dry mixing, in the next 30s half of the water was added, then in the next 30 s the other half of water and superplasticizer; finally, mixing was continued for an additional 2 min to achieve the most homogeneous mixture. After mixing, the concrete is poured into moulds measuring 15x15x15 cm, then compacted using vibrating table. After 24 h, the samples are removed from the mould and cured in water at a temperature of 20 ± 2˚C until the day of testing.

6 Results analysis and discussion

6.1 Workability of fresh concrete

The standard slump method [48] was used as a parameter for quantitative assessment of concrete workability, picture 11. From Table 2, it can be seen that all reference mixtures, regardless of the w/c ratio, and with a minimum amount of superplasticizers had very good workability (classes S4 and S5 according to [48]).

Mixtures with 30% cement replacement, in which the finest filler (L1) with a slightly higher amount of plasticizer was used, also had good workability, except for the mixture 30_L1_1 with the lowest w/c ratio at which the shear slump occurred. The mixture 30_L1_2 also had a shear slump, but it is realistic to expect that with the increase in the amount of superplasticizer (up to 2%), its workability will significantly improve and shear slump will be avoided. In a mixture in which a quartz filler was used, probably due to the excessive particle size, an improvement in workability cannot be expected, even with an increase in the w/c ratio to a maximum of 0.71 and a relatively high content of superplasticizers.

In mixtures where 45% of cement has been replaced by a filler, there is an additional deterioration in workability (desired slump class was not achieved), regardless of the filler used. In any case, the mixtures with L1 filler had far better workability than the others, so it can be expected that the application of a larger amount of superplasticizers could further improve their workability, which can be uneconomical. It may be better to look for a solution using a more powerful plasticizer. Larger fillers with large w/c ratios gave practically zero slump, while at lower w/c ratios the mixture is too dry.

It should be noted that the amount of paste in the volume of concrete was relatively low, 20-30%, and that all mixtures that had this value above 25% had a proper slump, so the possible solution to the workability problem may be to increase the powder component.

6.2 Compressive strength

The shape and dimensions of the specimens for determining the compressive strength must be in accordance with [50], while the method of production and curing process are defined in [51].

The compressive strength test is performed at the age of 28 days in accordance with [52], by gradual application of load (0.6 ± 0.4 MPa/s) to fracture. Three 15 cm cube samples were made for each mixture and the compressive strength was calculated as the mean of the measured values (f_{cm}). The test results are shown in Figures 12-17.

Figure 12 shows the dependence of the compressive strength of concrete and the w/c ratio. A significant decrease in strength with an increase in w/c is clearly observed. The percentage decrease in strength with increasing w/c ratio is shown in Figure 13 (the minimum value of w/c = 0.54 was taken as a reference).

After the regression analysis, excellent linear dependence can be observed, and high coefficients of determination (R^2 = 0.95 ÷ 0.99) confirm this, Figure 12. These lines are almost parallel. However, with the increase in the percentage of fillers, a slightly higher coefficient of direction was observed. Mixtures containing 30% of fillers have about 10% higher mentioned coefficient compared to the reference mixtures, while those with 45% replacement have this coefficient higher by about 18 %. That indicates a slightly greater reduction in the strength of mixtures in which the cement is partially replaced by a filler (in relative terms, due to the higher strength of these samples, the percentage drop is smaller).

On average, due to the increase in the value of w/c ratio for every 0.1, there is a decrease in strength by about 8.5, 9.5, and 10 MPa, for mixtures with 0, 30, and 45% of replaced cement.

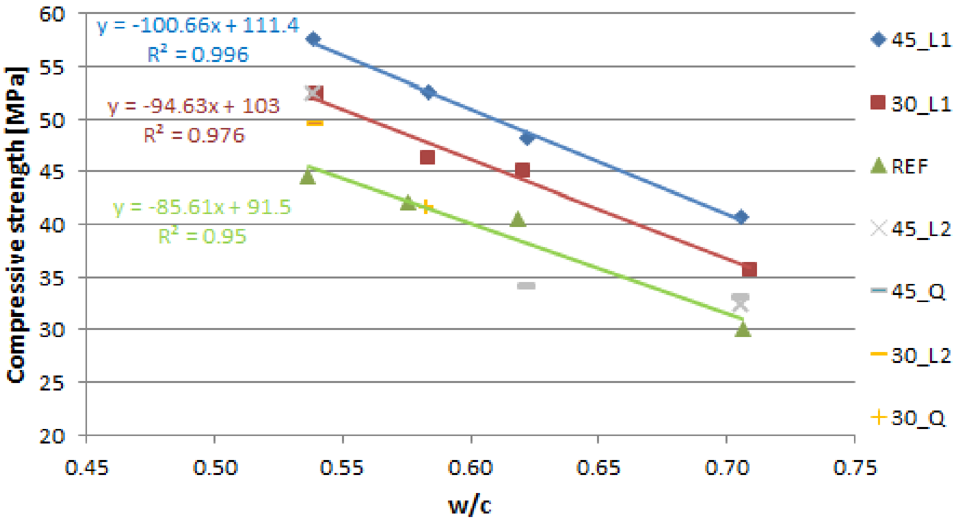

Figure 12. Compressive strength as a function of w/c ratios and percentage of cement replacement

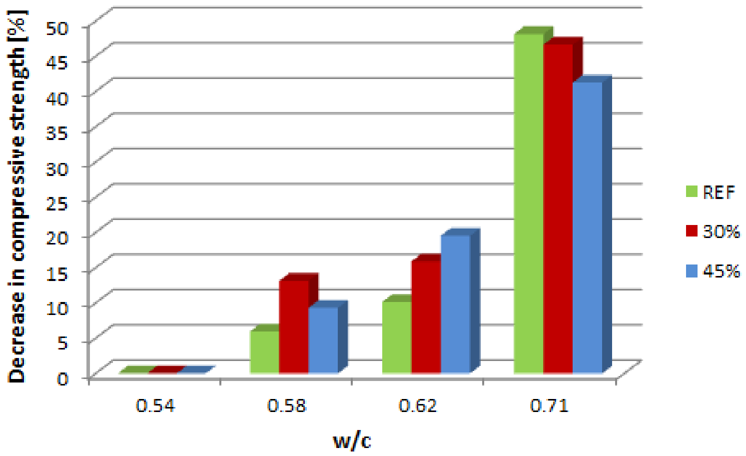

Figure 13. Influence of w/c ratio: reduction of compressive strength for different w/c ratios expressed in percent, filler L1

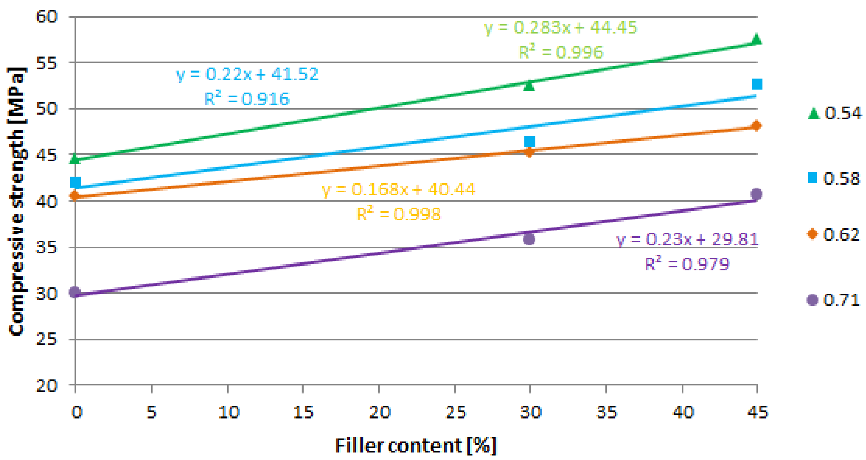

Figure 14. Compressive strength as a function of cement replacement percentage, for different w/c ratios, filler L1

In the reference samples, compared to the highest strength (44.6 MPa) obtained for the lowest w/c ratio of 0.54, a decrease in strength of 6, 10 and 48% was recorded, for w/c ratios of 0.58; 0.62, and 0.71, respectively. In mixtures in which 30% of cement is replaced by the finest filler (L1), these reductions are 13, 16, and 46%, respectively, while in the case of 45% replacement these values are 9, 19, and 41% for the same w/c ratios.

In order to understand better the influence of fillers, Figure 14 shows the dependence of compressive strength of mixtures with filler L1 as a function of the percentage replacement of cement for different w/c ratios. With a decrease in the amount of cement, i.e. with an increase in the filler in the mixture, a linear trend of increasing compressive strength for all w/c ratios is observed. On average, for every 10% of cement replacement, the strength increases by about 2.3 MPa. In this case, too, a high level of correlation was achieved (R^2> 0.9).

The increase in strength as a function of the percentage of cement replacement is shown in Figure 15. The figure shows an average increase in strength of 14.7% in the case of mixtures in which 30% of cement was replaced by filler, while in mixtures with 45% of cement replacement an average increase of approximately 27% was recorded.

The increase in compressive strength is a consequence of the improved particle packing density, which is to some extent reflected in the increase in the density of the samples, Figure 16. The figure shows that there is a certain correlation, but the density is a rather sensitive and insufficiently reliable parameter. Nevertheless, it can be used for possible preliminary assessment.

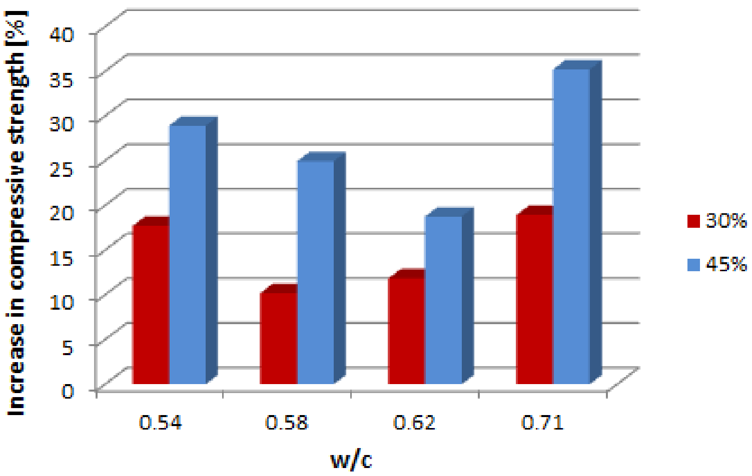

Figure 15. Influence of replacement percentage: increase in compressive strength in relation to the reference mixture depending on the replacement percentage of cement, filler L1

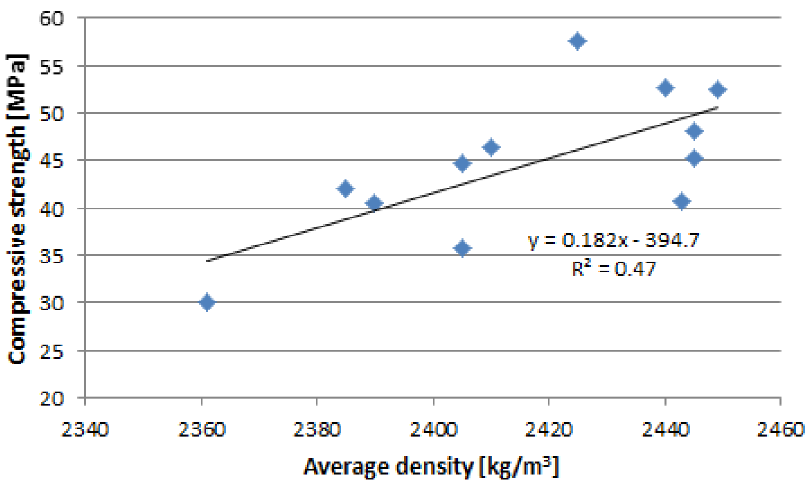

Figure 16. Compressive strength as a function of concrete density

Analysing the samples in which larger fillers were used (L2 and Q), it was found that their strengths are lower compared to the strengths of the samples in which the finest filler (L1) was used, Figure 17. Thus, the compressive strength of concrete decreases compared to reference samples with increasing filler size, regardless of the w/c ratio and the percentage of cement replacement. In the case of L2 filler application, a decrease of 6-25% was observed. In the case of samples with quartz filler, in some cases, the decrease in strength was up to 40%. The effect of filler size seems to be less pronounced in mixtures with a lower percentage of cement replacement.

It should be borne in mind that these conclusions were drawn on the basis of relatively limited number of samples, and in order to analyse in more detail the influence of filler size and to draw more reliable conclusions, it is desirable to expand the scope of experimental tests. It should be noted that larger fillers are significantly cheaper (4-5 times), so if high concrete compressive strength is not required, their application can be economically justified, provided that the problem of insufficient workability is solved.

7 Conclusions

In this study, green concrete mixtures were made for structural use. In the scope of the extensive test program, the influence of different percentages of cement replacement, water-cement ratio, but also the particle size of the applied filler, was analyzed.

The conducted analyses undoubtedly show the great potential of green concrete with low cement and high filler content and their potential application in reinforced concrete structures. Experimental tests have shown that it is possible to make concrete of the usual strength classes (C25/30; C30/37) and even higher, with a significantly reduced amount of cement (<200 kg/m^3). In this way, large economic savings can be achieved and the negative impact of concrete industry on the environment can be significantly reduced, especially if we take into account the amount of cement produced, which is constantly increasing.

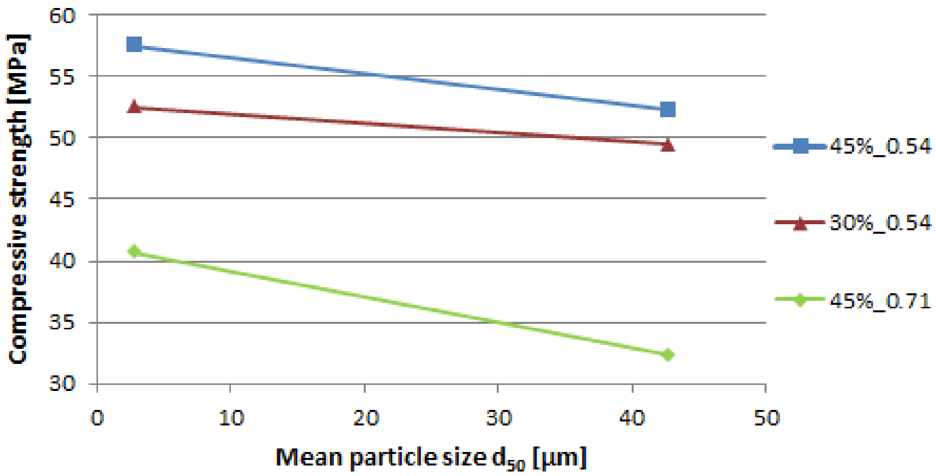

Figure 17. Influence of filler size on compressive strength of concrete

Based on the experimental results, the following conclusions can be drawn:

• One of the most important parameters in this type of concrete is the w/c ratio, which plays a key role in most properties of concrete in the fresh and hardened state;

• The reduction of binder content often may lead to the reduction in compressive strength. In order to preserve the high compressive strength, it is necessary to keep the same w/c ratio in relation to the reference mixture;

• It should be kept in mind that due to the decrease in the value of w/c ratio for every 0.1, there is a linear increase in strength on average about 9 MPa in all samples, and the increase is slightly smaller in mixtures with higher percentage of cement replacement;

• The application of one of the models to improve the particle packing density is very desirable, and even fairly simple models such as the model proposed by Funk and Dinger provide very good results;

• With the increase of the percentage of cement replacement, at the same w/c ratio, the compressive strength of concrete increases, which is the consequence of the increase of particle packing, however, the workability of mixtures is disturbed, which is very important for practical application;

• On average, for every 10% of cement replacement, the strength increases by about 2.3 MPa. Thus, in samples with 30% of replaced cement, an increase of 14.7% was observed on average, while in those with 45% of replaced cement, an average strength increase of approximately 27% was recorded;

• Great attention should be paid to solving the problem of workability. Therefore, the use of new generation superplasticizers and possible increase in the content of powder component should be pointed out as potential solutions, which can be achieved by simply adopting a lower exponent value in the Funk and Dinger model. The combination of these two proposals can give an optimal solution. The use of second-generation superplasticizers contributes to the improvement of workability only to a certain extent, and overdosing can be economically unprofitable.

• If it is not necessary to obtain high-strength concrete, workability can be improved more simply and economically by correcting the w/c ratio. In contrast, if it is necessary to achieve high compressive strengths of concrete, it is possible to use cement of finer grades and possibly higher class, with a reduction in w/c ratio;

• When it comes to filler size, preference should certainly be given to finer fillers that, regardless of the w/c ratio, have a positive effect on workability, but also on strength (in some cases an increase of up to 40%). This effect is more pronounced with higher percentage of cement replacement;

• The proposed equations obtained by regression analysis can be used for practical assessment of the influence of certain parameters on the compressive strength of this type of green concrete;

• Further research should focus on forming a database to draw more detailed conclusions and suggest more reliable equations.

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Tecnical Paper

Physical-mechanical properties and durability of Ultra-high Performance Concrete (UHPC)

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ABSTRACT

This paper presents the results of the authors' laboratory testing of physical, mechanical and durability properties of Ultra-high Performance Concrete (UHPC). The short history of development and application of UHPC concrete is presented in the first part of this paper while the second part deals with the experimental investigation, presenting the results of material characterization obtained from physical-mechanical and durability tests. Based on the results shown in the paper, the mean value of compressive strength obtained at 28 days is 114 MPa, with the average density of 2270 kg/m3 in hardened state. The results showed that tested UHPC belongs to the highest class of water impermeability V-III, as well as the highest class MS0 (without visible damage) in a simulated freeze-thaw environment and de-icing salt attack test. Also, the highest class XM3 for abrasion resistance was achieved. Additional tests showed that the tested concrete fulfils the requirements for the highest exposure classes XC4 and XD4, in terms of resistance to carbonation and the penetration of chloride ions. Conclusions and recommendations for further development and possible application of UHPC are presented at the end of paper.

1 Introduction

Ultra-high Performance Concrete represents one of the most innovative and promising directions of concrete development in modern structural engineering. The reason for the fast development of concrete properties is a rapid growth of the civil engineering industry (Figure 1) [1]. In this context a substantial increase in construction of high rise buildings and long spans bridges within a short time period can be noticed[2]. This is the main reason for a significant enhancement in physical-mechanical properties and the durability of concrete. In the last 30 years, the durability of concrete structures has become a priority in well-developed countries, because 30-50% of the total budget for construction works has been spent on repairs and maintenance of existing structures [3]. A goal to minimize repair costs has led to a remarkable improvement in the durability and impermeability (resistance to harmful environmental agents) of UHPC which in turn translates to reduced maintenance costs and a longer life span of the structures.

UHPC concrete fulfils all the specific requirements of modern civil engineering and therefore has found multiple applications in modern construction in the last 30 years. A great number of technical papers and studies have been published on UHPC concrete, regarding development of its properties and areas of application. In the initial developing phase, UHPC concrete was made with high quality natural materials like cement CEM I (pure portland cement clinker), quartz sand, quartz flour, silica fume, HRWR agents (High Range Water Reducer -super plasticizer), steel fibres. The third intensive source of global CO_2 emission is the cement industry with app. 5% share (Figure 2)[4], [5]. Also, exploitation of quartz sand is endlessly ruining natural resources and it is harmful for the environment. In the last 10 years there is a trend to use cheaper and recycled materials (fly ash, ground granulated blast furnace slag, limestone powder) [6], [7], [8] as a partial cement replacement in UHPC mix designs, as well as partial replacement of quartz sand and quartz flour with waste glass [9].

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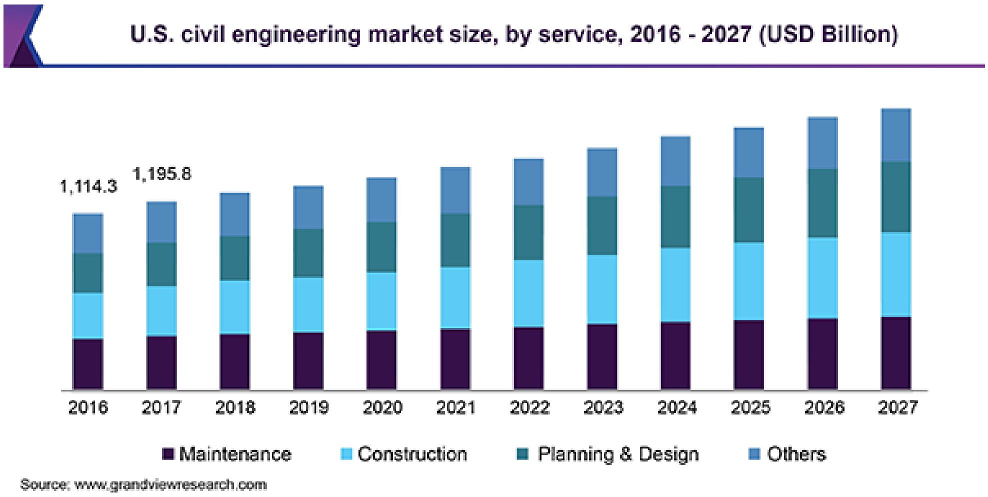

Figure 1. Civil engineering market size in the USA 2016-2027 [11]

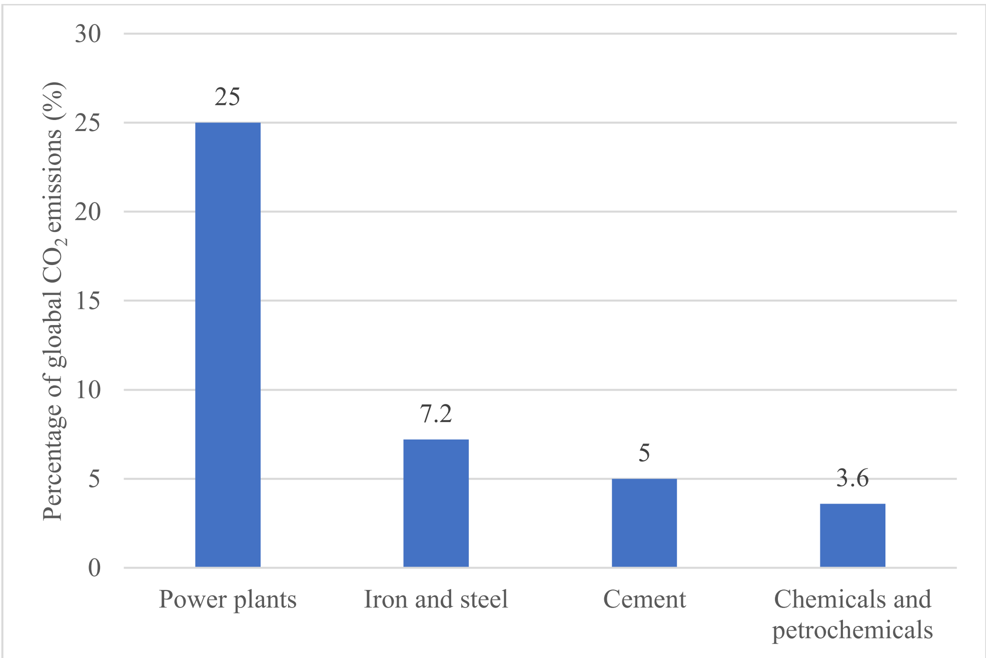

Figure 2. Trend of CO_2 emissions by worldwide industry [5]

2 Ultra-high performance concrete in general

2.1 Definition of Ultra-high Performance Concrete (UHPC)

The most commonly used definition of UHPC is new generation cementitious composite with improved physical-mechanical characteristics, durability and ductility compared to normal concrete and High Performance Concrete (HPC) [10].Current codes, standards and guides already consider parameters for specification of UHPC.

The most important of these parameters are: compressive strength of at least 120 MPa at 28 days, tensile strength of at least 20 MPa at 28 days and high durability and ductility with a very low water/cement ratio.

In several of the recently published papers the authors investigated UHPC concrete with fibres as a combination of three concrete technologies: self-compacting concrete (SCC), fibre reinforced concrete (FRC) and high-performance concrete (HPC) (Figure 3)[11],[12], [13].

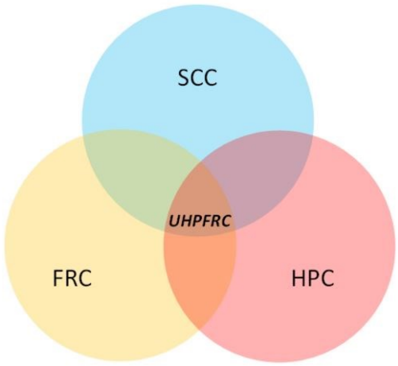

Figure 3. Definition of UHPFRC concrete (UHPC concrete with fibres)[12]

2.2 Properties of Ultra-high Performance Concrete (UHPC)

UHPC is a relatively new material, used for 30 years in construction, and in experimental programme for 40 years. The most important properties of UHPC are:

– UHPC is typically made with large amounts of Portland Cement CEM I (from 700 to 1000 kg/m^3) and with a very low w/c ratio [6], [8], [9], [10], [11], [13], [14], [15];

– This concrete has a high compressive strength (over 120 MPa at 28 days), as well as tensile strength (over 20 MPa at 28 days) [6], [8], [9], [10], [11], [13], [14], [15];

– UHPC is usually mixed like a self-compacting concrete (SCC), which means superior fresh state properties in terms of consistency and workability;

– Improved durability, mainly resistance to carbonation and chloride ion penetration which provide longer service life of structures; also, UHPC has a high-level resistance to water penetration, freeze and salt attack, as well as abrasion resistance;

– Using steel fibres in the mixture helps in strengthening of the cement matrix which exhibits significantly better mechanical characteristics, and also higher ductility capacity of structural elements in seismic area;

– Using UHPC gives the opportunity for reduction in cross sections of structural elements and total costs.

All of the above stated are positive properties of UHPC which were a motivation for further development and application of this construction material. However, UHPC also has some negative characteristics, such as:

– High amount of cement in the mixture with low w/c ratio gives poor rheological properties (shrinkage and creep). Furthermore, a large proportion of cement cannot be completely hydrated with relatively low w/c ratio. This is not eco-friendly and not cost-effective[7];

– Lack of standards and codes for design and numerical modelling of UHPC structures. At the moment, some countries, like France, have guidelines for the design of mixtures and properties of UHPC in a fresh and hardened state [16] and standards for design of concrete structures from non-reinforced, reinforced and prestressed UHPC with or without steel fibres [17];

– Production of UHPC includes increased costs of mixtures, workforce and construction. Qualified workforce and special equipment are required for construction of UHPC structures.

2.3 Application of Ultra-high Performance Concrete (UHPC)

UHPC is rapidly taking an important place in modern engineering. For the last 30 years, UHPC has been used in various applications in civil engineering from buildings to bridges. UHPC can be successfully used for production of precast bridge girders, bridge deck slabs, seismic columns, wind turbine masts, tunnels, piles etc. Besides, UHPC can be used for repair and reconstruction of existing structures after accidents and damages, or the expired life cycle of the structures.

The most important application of UHPC is in bridge construction. Over 40 bridge structures have been completed using UHPC, most of them in the USA, Canada and China[18]. The first application of UHPC in North America was in 1997, for construction of the “Sherbrooke Pedestrian Bridge“(Figure 4) in Quebec, Canada [10]with a clear span of 60 m. The first UHPC bridge in the USA was a highway bridge “Mars Hill Bridge” (Figure 5) with a total span of 33.5 m [19], [20].The span record-holder is a pedestrian bridge “The Peace” made in 2002 in Seoul, South Korea with the main arch span of 120 m [10]. The pilot project for bridge construction in China was a railway bridge “Luan Bai Dried” built in 2006 [21]. In this context, UHPC has been used to ensure connection performance in bridge joints [18].

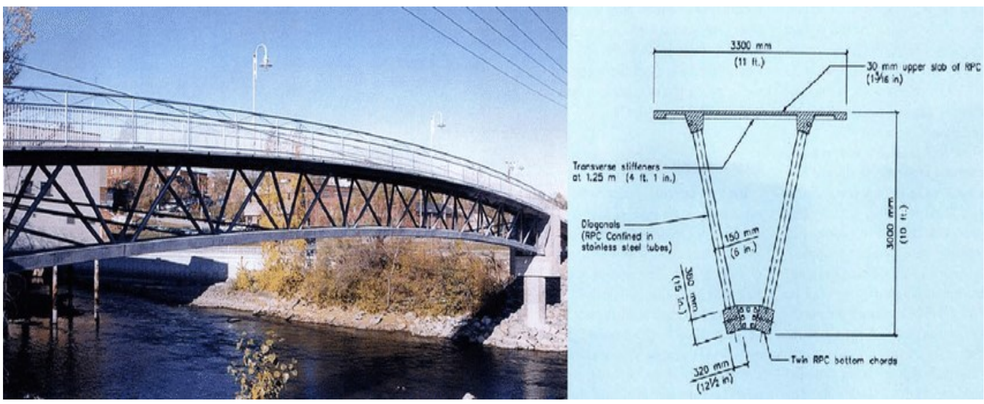

Figure 4. Sherbrooke Pedestrian Bridge, Quebec, Canada (1997) [22]

Figure 5. Mars Hill Bridge, Iowa, USA (2006) [20]

UHPC has also gained interest in the field of building components, such as roof components, cladding, sun shades, lattice façade elements and others. The MuCEM (Figure 6) is the first building in which UHPC was used in different context - as façade elements and cladding [10]. The first application of UHPC in Spain was in 2003 for the composite structural elements of the Reina Sofia Museum in Madrid (Figure 7). Commercial UHPC known as Ductal was imported from France to manufacture the composite columns [10].

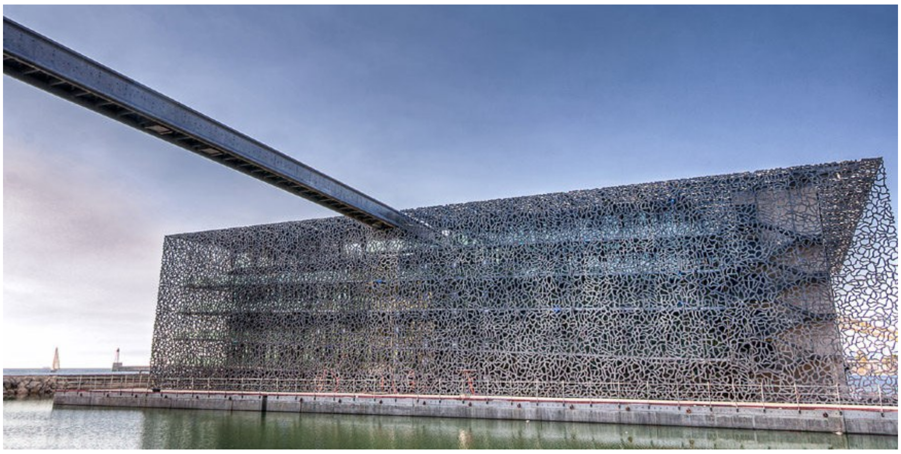

Figure 6. MuCEM museum, Marseille, France (2013) [23]

Figure 7. Reina Sofia Museum in Madrid, Spain (2013) [24]

3 Experimental programme

3.1 Scope of the experimental programme

Laboratory testing included methods to determine the basic physical-mechanical properties (density and compressive strength), water tightness, resistance to freeze-thaw and de-icing salt attack, test of abrasion resistance and additional durability parameters (resistance of the samples to carbonation and penetration of chloride ions).

Examination of UHPC was performed on cubes with dimensions of 10x10x10 cm (compressive strength test) and 7.07x7.07x7.07 cm (abrasion resistance test), as well as on prisms with dimensions of 4x4x16 cm(carbonation test), plates 10x10x5 cm (water impermeability test and freeze-thaw and de-icing salt attack test)and cylinders with a diameter/height of 100/100 mm (chloride ion penetration test).The samples were tested at the age of 1.3 and 28 days according to [25] and [26]. Number of samples is 3 for each of considered test.

3.2 Mixture design

Only one mixture with cement CEM I 52,5R (C), microsilica (MS),coarse quartz aggregate 300-500µm (CQ), fine quartz aggregate 150-300µm (FQ), High Range Water Reducer (HRWR) admixture and water (W) was considered. List of used materials with type, producer and specific density is presented in Table 1. Table 2 shows mixture proportions in relation to the cement amount, whose quantity was 850 kg/m^3 in this research. Mixture was designed according to recommendations from literature and standards for UHPC. The mix was further improved through software model for packing system optimization similar to Andersen & Andersen model. Rapid mixer with volume of 10 litres was used for mixing. Procedure of mixing is explained in Figure 8. Concrete specimens were subjected to wet curing regime, for 24 h under water-soaked cloth.

3.3 Results of the experimental programme

3.3.1 Basic physical-mechanical properties

The basic physical-mechanical characteristics of the tested materials (density, compressive strength) were obtained using the standard methods by testing cubes and prisms with dimensions of 10x10x10 cm and 4x4x16 cm, respectively. At the time of testing, the specimens were 24 hours, 3 days and 28 days old. The test results are shown in Table 3 and Table 4. The compressive strength was tested according to SRPS EN 12390-3:2014[27] and density according to SRPS EN 12390-7:2009 [28].

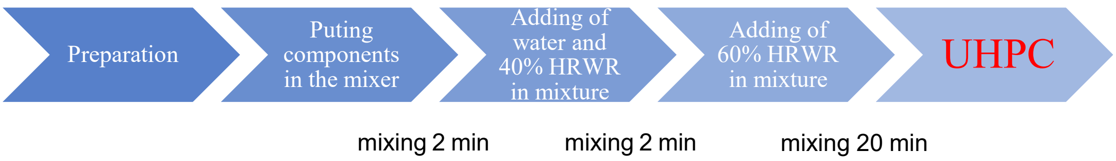

Figure 8. Mixing procedure

Table 1. List of used materials

Material	Type	Specific density (kg/m^3)
Cement	CEM I 52,5R	~ 3150
Sand	Quartz sand	2640
Silica fume	Micro-silica	2200
Admixture	HRWR	1050
Water	Potable water	1000

Table 2. Mixture proportions

Material	C	MS	CQ	FQ	HRWR	W
Proportion	1.000	0.184	1.043	0.337	0.046	0.220

Table 3.Test results of compressive strength

Sample number	Compressive strength (MPa) –age 24 hours	Compressive strength (MPa) –age 3days	Compressive strength (MPa) –age 28days
1	82.4	91.2	-
2	83.2	87.7	115.9
3	90.0	85.7	112.1
Average	**85.2**	**88.2**	**114.0**

Table 4.Test results of concrete density

State	Average density (kg/m^3)
Fresh	**2281.9**
Hardened (after 24h)	**2270.2**

3.3.2 Results of water impermeability test

The test of water penetration under pressure was carried out according to SRPS EN 12390-8:2010 [29]. Examination of water penetration was performed on plate specimens with dimensions of 10x10x5cm at the age of 28 days. The test results are shown in Table 5.

3.3.3 Results of freeze-thaw and de-icing salt attack test

The investigation of the UHPC resistance to frost and de-icing salt attack was carried out according to the standard SRPS U.M1.055:1984[30]. The investigation was carried out on plate samples that measured 10 x10 x5cm at the age of 28 days in a chamber with regulated temperature and humidity conditions. No damages after 25 cycles were observed on the surface of the samples as it can be seen on Figure 9.

Table 5. Test results of water penetration

Sample	**V1**		**V2**		**V3**	
Side	Left	Right	Left	Right	Left	Right
Depth [mm]	5.5	5.0	4.4	4.3	8.9	9.5
Average depth [mm]	**6.3**					

Figure 9. Surface of concrete sample after 25 cycles of freeze-thaw and salt attack

3.3.4 Results of abrasion resistance test

The investigation of the abrasion resistance of UHPC was performed by using a test according to SRPS: B.B8.015:1984 [31]. The used samples were 28 days old and square in shape, with the abrasion surface of 50 cm^2. The test results are shown in Table 6.

Table 6. Results of abrasion resistance test

Sample number	**Volume mass**	**Loss mass m [g]**	**Abrasion wear H_B[cm^3/50 cm^2]**
1	2.29	24.00	**10.60**
2	2.34	24.90	**10.60**
3	2.33	23.20	**9.70**

3.3.5 Additional durability parameters

3.3.5.1 Resistance to carbonation

The investigation of the resistance to carbonation was completed using an accelerated test according to 8. fib Bulletin No.34: Model Code for Service Life Design, fib (2006) [32]. The samples used for this test were prisms, broken at the age of 28 days. The samples were put into a chamber in which they were exposed to CO_2 with a concentration of 2% for the next 28 days at the constant temperature of 20°Cand humidity of 65%. After this period of time, the samples were split apart and tested using phenolphthalein solution C20H1404 and left to dry for 30 mins. After the aforementioned time, the treated sample surfaces were entirely pink, which indicates the following: there is no decrease in the alkalinity of the concrete, specifically the depth of carbonation was measured at 0.00 mm. As a conclusion, the rate of carbonation after the accelerated test has been found negligible

Figure 10. Appearance of the sample surface after treatment with phenolphthalein solution test

3.3.5.2 Resistance to chloride ion penetration

The investigation of the resistance of chloride ion penetration was performed using concrete cylinder specimens with diameter100mm and 100mm long, at the age of 28days. The test was carried out according to the standard NT Build 492: Non-Steady State Chloride Migration, Nordic Council of Ministers (1999) [33]. After the treatment with silver nitrate solution, the penetration of the chloride ions in the form of a layer of silver colour on the surface of the sample could clearly be seen on the treated surface. The chloride penetration was measured at seven points, excluding edges, where the values obtained were used to determine the chloride ion migration coefficient. The test results are shown in Table 7.

4 Discussion of results and conclusions

Ultra-high Performance Concrete represents a new generation of construction material which has been developed rapidly in the last twenty to thirty years. A substantial number of papers covering this composite material has been published recently in scientific and expert journals. Based on the results of the laboratory tests conducted, the following conclusions can be made:

Table 7. Results of chloride ion penetration resistance test

Sample number	Voltage [V]	L_1[mm]			l_{30}	t_p	t_k	t[h]	x_{av}[mm]	D_{nssm}[m^2/s]
1	60	51.0			1.8	18.6	19.1	120	2.6	**$10.002 \cdot 10^{-14}$**
2	60	51.5			1.7	18.6	19.1	120	1.8	**$6.888 \cdot 10^{-14}$**

Figure 11. Appearance of the sample surface after treatment with silver nitrate solution

– First, the concrete density is lower than in normal concrete without and with reinforcement bars (2400 and 2500 kg/m^3 respectively, for normal concrete). The effect of this is a lower dead weight of structures made with UHPC compared to typical concrete structures. This results in reduced design forces and allows smaller dimensions of structural elements. Testing of modulus of elasticity was not in the scope of this investigation, but according to relevant literature this property should be higher than 40 GPa. [34];

– Based on results shown in Table 2, the mean value of compressive strengths obtained at 1 day is 85.2 MPa, at 3 days 88.2 MPa and at 28 days 114.0 MPa lower than 120 MPa (recommended value from standards), but it should be noted that is preliminary testing of premix. This means that the compressive strength of UHPC is significantly larger (3-4times) than normal concrete. Again, this allows smaller dimensions of structural elements made of UHPC;

– The mean value of water penetration was measured at 6.3 mm and it can be concluded that the tested UHPC belongs to the highest class V-III (very low water permeability) as per SRPS EN 12390-8:2010 [29]. This concrete is suitable for structures in highly aggressive environments which will be in contact with water, like dams, river and marine bridges (piles and columns in water), etc.;

– After the freeze and de-icing salt test, no damages were observed on the surface of the samples, which means that the UHPC belongs to the highest class MS0 (zero damage) according to SRPS U.M1.055:1984 [30].Based on this, the UHPC is suitable for structures exposed to severe weather conditions (like concrete slabs of bridges);

– Based on the results shown in Table 5, the average value of abrasion wear H_B was less than14 cm^3/50cm^2, meaning that the UHPC belongs to the highest class XM3 according to SRPS: B.B8.015:1984 [31].

– After the accelerated carbonation test, no decrease in the alkalinity of the concrete was observed, i.e. the depth of carbonation was measured to be 0.00 mm. The tested concrete belongs to the highest class of carbonation resistance, which corresponds to the exposure class XC4 – as defined by the standard SRPS EN 206:2017 [35];

– The mean value of the chloride migration coefficient was 8.454*10^{-14} m^2/s. According to this result, the resistance of the tested concrete is 8-10 times higher than ordinary concrete, which classifies the UHPC in the group “very good”, i.e. in the highest exposure class XD4 as defined by SRPS EN 206:2017 [35].

Based on all the above stated results, the general conclusion is that UHPC has significantly increased physical-mechanical properties and improved durability when compared to the normal concrete for usual applications. Increased strength (both compressive and flexural) gives the opportunity for reduction of structural element dimensions and avoidance of classic reinforcement bars (steel fibres might be required if necessary). In addition, improved durability gives us the opportunity to use UHPC in structures which are exposed to severe deterioration mechanisms (i.e. harmful environmental agents), with a prospect of a much longer service life.

Main disadvantage of UHPC is the requirement for a large quantity of pure portland cement CEM I (700-1000 kg/m^3) which is especially harmful for the environment (app. 5% of global emissions of CO_2 comes from the cement industry and also CEM I is very expensive component in the mixture). Such a large amount of cement in the concrete mix makes its rheological characteristics (shrinkage and creep) worse.

Opportunities for further development of UHPC lay in the mix design optimization, where some amount of CEM I will be replaced by mineral additions like fly ash, rice husk ash, blast furnace slag, silica fume and others. In addition, it is possible to replace some amount of quartz sand and filler with recycled waste glass. This is the way for development of UHPC in the future with better properties, lower construction price and less harmful environmental effects.

Further development is also possible related to the standards and codes for design of UHPC structures, which will promote UHPC as a contemporary structural material and increase its application potential in civil engineering.

In further studies and research work planned for near future, investigation on the durability of UHPC concrete with steel fibres will be considered as well as experimental investigation of mixture with partial cement replacement and use of recycled waste glass instead of fine quartz and quartz sand. Other tests like DTA-TGA, XRD, ITZ, SEM analysis and further physical-mechanical properties of the UHPC (modulus of elasticity and hydration rate) will be considered in future research as well.

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Technical Paper

New Eurocode 2 provisions for recycled aggregate concrete and their implications for the design of one-way slabs

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ABSTRACT

A significant amount of research has been performed on recycled aggregate concrete (RAC), both on the material and structural level. This has enabled the formulation of material and structural resistance models that can be safely and reliably used for the structural design of RAC members and the new Eurocode 2 (EC2) will contain an informative annex detailing provisions for the design of RAC. Thus, an increased market uptake of recycled aggregate (RA) can be achieved, leading to potential sustainability improvements of concrete structures. In order to familiarize designers with the new provisions for RAC, this paper presents an example of one-way slab design using varying RA substitution ratios, as well as a parametric study on the implications of RAC provisions on slab slenderness. The results of this study show that RAC one way slabs can be successfully designed using EC2. Although such slabs might require larger depths than natural aggregate concrete slabs, their applicability in the typical slenderness range is possible.

1 Introduction

The movement towards increasing sustainability of all aspects of human activity is gaining momentum, from the UN 2030 Agenda [1] to the European Green Deal [2]. Among these initiatives, achieving the sustainability of the construction industry is critical, considering its global environmental impact and social importance [3,4]. This goal can be reached by transforming the construction sector into full circularity [5].

Within these plans, the "greening" of concrete is crucial considering its production of over 25 billion tons per year [6]. One important aspect is the use of aggregates in concrete production: on the one side, over 40 billion tons of natural aggregate (NA) are produced annually [7] and, on the other side, construction and demolition waste leave behind immense quantities of waste [8]. Therefore, an option that has gained importance and helps address both challenges is the recycling of construction and demolition waste (CDW) to produce recycled aggregate (RA) which can later be used to replace NA and produce recycled aggregate concrete (RAC).

The use of RA to produce structural RAC has been heavily researched over the past decades and significant knowledge of material and structural behavior of RAC has been generated. In order to leverage this knowledge and increase the market uptake and implementation of RAC, new structural design standards that provide design guidelines for RAC are necessary.

In Europe, the European Standardization Organization (CEN) has the mandate to develop structural design codes (Eurocodes). Currently, a new version of Eurocode 2 prEN1992-1-1 (EC2) for the design of concrete structures is being developed and will contain an annex with special provisions for the structural design of RAC members [9]. Such an innovation will enable designers to safely and reliably design and construct RAC structures.

Therefore, it is first necessary to familiarize the users of EC2 with these new provisions and assess their implications on the design of RAC members. For this purpose, a calculation example and parametric analysis of one-way reinforced RAC slab design is presented. One-way slabs are chosen as very common elements in building structures, that are typically governed by serviceability criteria which are of great importance in RAC design. Hence, first the provisions of EC2 for RAC are presented, after which one-way slab ultimate limit state (ULS) and serviceability limit state (SLS) design is presented and discussed, followed by concluding remarks.

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2 Annex N of prEN1992-1-1

The provisions for RAC in EC2 are given in Annex N. An extensive background to Annex N can be found in the study by Tošić et al. [10], wheras herein it will be only briefly presented. Namely, as a CEN standard, EC2 is normatively linked to other standards, in this case the standard for concrete EN 206 [11] and for aggregates for concrete EN 12620 [12].

Firstly, EN 12620 presents a composition-based classification of coarse RA (particle size > 4 mm) into aggregates composed of crushed concrete (*Rc*), unbound stone (*Ru*), crushed brick (*Rb*), bituminous material (*Ra*), glass (*Rg*), floating material (*FL*) and other (*X*). The standard also provides a nomenclature for RA based on the percentages of each component type. For example, Rc_{90} is RA that contains ≥90% of crushed concrete (*Rc*), whereas Rb_{30-} is RA that contains <30% of crushed brick (*Rb*).

Secondly, EN 206 classifies coarse RA into two types: Type A and Type B, based on their composition:

- Type A (Rc_{90}, R_{cu95}, $R_{b10\text{-}}$, $R_{a1\text{-}}$, $FL_{2\text{-}}$, $XR_{g1\text{-}}$) and
- Type B (Rc_{50}, R_{cu70}, $R_{b30\text{-}}$, $R_{a5\text{-}}$, $FL_{2\text{-}}$, $XR_{g2\text{-}}$)

Further, EN 206 provides certain limits for coarse RA incorporation based on environmental exposure classes (e.g. 50% for X0). However, these limits are imposed so that no change of RAC properties is assumed relative to a reference natural aggregate concrete (NAC). Therefore, what is needed are limits on RA substitution that are accompanied by material and structural models that take into account the effects of RA incorporation. In this way, the provisions of Annex N were formulated.

The basic variable that defines the effect of RA on RAC properties is the RA substitution ratio α_{RA} defined as the "quantity of fine and coarse recycled aggregate/total quantity of aggregate" (i.e. varying from 0 to 1). This way, the incorporation of fine RA (particle size ≤ 4 mm) with future revisions of EN 206 and EN 12620 is also facilitated.

For lower RA substitution ratios ($\alpha_{RA} \leq 0.2$ for reinforced concrete, RC) it is assumed that there are no changes to RAC properties. For higher RA substitution ratios ($0.2 < \alpha_{RA} \leq 0.4$ for RC, $\alpha_{RA} \leq 0.2$ for prestressed concrete, PC), special provisions given in Table 1 may be applied. For even higher substitution ratios ($\alpha_{RA} > 0.4$ for RC and $\alpha_{RA} > 0.2$ for PC) properties of RAC should be measured. All of the above stated is valid for Type A RA (according to EN 206 [11]), whereas for Type B, the limiting substitution ratios should be decreased by 50%.

As can be seen from Table 1, the largest impact of RA incorporation is on elastic and long-term RAC properties, i.e. modulus of elasticity, shrinkage and creep. On the structural level, beside an effect on the reduction of the roughness zone in shear cracks, there is a reduction of tension stiffening that impacts deflection control. Finally, for durability, the new EC2 allows either the use of current environmental exposure class-based concrete covers or the use of a new performance-based classification based on "exposure resistance classes" (ERC). The use of ERCs means that concrete cover is directly determined based on the concrete resistance class (determined by testing or deemed-to-satisfy requirements on concrete mix design). Therefore, in this case, no changes in cover are necessary between NAC and RAC that belong to the same ERC. However, as countries will be able to opt out of the use of ERCs, provisions for increasing the minimum concrete cover due to durability are provided for exposures to carbonation and chloride ingress.

3 Design of one-way reinforced RAC slabs according to Annex N

3.1 Description of one-way reinforced RAC slab parametric study

To analyze the implications of Annex N provisions on RAC design, a simply supported one-way slab was selected (considering a 1-m wide strip, i.e. b = 1000 mm). The span of the slab L was constant and equal to 6 m. The effective depth d was determined by considering L/d ratios from 15 to 25. In this way, the range of slenderness limits prescribed by prEN1992-1-1 for one-way elements was covered. In this way, effective depths ranged from 240 to 400 mm.

The slab was considered as an indoor element in a building structure. Hence, an environmental exposure class XC3 was assumed and a nominal cover $c_{nom} = c_{min,dur}$ equal to 20 and 25 mm for NAC and RAC, respectively. Then, the reinforcement center of gravity d_1 was adopted as 30 and 35 mm for NAC and RAC, respectively and the overall height $h = d + d_1$. A relative humidity of 50% was adopted.

Two concrete classes were considered: C25/30 and C50/60 to cover both extremes of RAC applicability according to Annex N. Finally, three concretes were considered for each class: α_{RA} = 0, 0.2 and 0.4, i.e. NAC and RAC at the lower and upper limit of RA substitution ratios compatible with provisions in Table 1. Herein, the concretes were denominated NAC, RAC 0.2 and RAC 0.4.

The load on each element consisted of self-weight (determined from slab thickness and considering the adjustment for RAC in Table 1), additional dead load of 1.5 kN/m^2 and a live load of 3.0 kN/m^2, with a quasi-permanent combination coefficient ψ_2 = 0.3. Reinforcement B500B was considered and a service life of 50 years. In total, 66 cases were generated (11 L/d ratios per concrete, 2 concrete classes and 3 RA substitution ratios).

3.2 Comparison of the design of RAC and NAC one-way slabs

First, ULS design was performed on each slab by calculating the necessary longitudinal reinforcement for ensuring flexural resistance, $A_{s,ULS}$. In this part of design, there are practically no differences between NAC and RAC. Although RAC slabs are 5 mm thicker than NAC ones (for a given L/d), this increase is offset by a reduced self-weight so that the differences in bending moments and $A_{s,ULS}$ reinforcement do not exceed 1%.

Table 1. Proposed expressions for RAC design properties.

RAC property	Correction for RAC
Density	$\rho_{\mathrm{RAC}} = 2.50 - 0.22 \cdot \alpha_{\mathrm{RA}}$
Modulus of elasticity	$E_{\mathrm{cm}} = k_{\mathrm{E}} \cdot (1 - 0.25 \cdot \alpha_{\mathrm{RA}}) \cdot f_{\mathrm{cm}}^{1/3}$
Shrinkage strain	$\varepsilon_{\mathrm{cs,RAC}} = (1 + 0.8 \cdot \alpha_{\mathrm{RA}}) \cdot \varepsilon_{\mathrm{cs}}$
Creep coefficient	$\varphi_{\mathrm{RAC}} = (1 + 0.6 \cdot \alpha_{\mathrm{RA}}) \cdot \varphi$
Peak strain	$\varepsilon_{\mathrm{c1}} = (1 + 0.33 \cdot \alpha_{\mathrm{RA}}) \cdot 0.7 \cdot f_{\mathrm{cm}}^{1/3} \leq 2.8‰$
Ultimate strain	$\varepsilon_{\mathrm{cu1}} = (1 + 0.33 \cdot \alpha_{\mathrm{RA}}) \cdot [2.8 + 14 \cdot (1 - f_{\mathrm{cm}}/108)^4] \leq 3.5‰$
Shear strength	$\tau_{\mathrm{Rd,c}} = (1 - 0.2 \cdot \alpha_{\mathrm{RA}}) \cdot \frac{0.66}{\gamma_{\mathrm{V}}} \cdot \left(100 \cdot \rho_{\mathrm{l}} \cdot f_{\mathrm{ck}} \cdot \frac{d_{\mathrm{dg}}}{d_{\mathrm{v}}}\right)^{1/3}$ $\tau_{\mathrm{Rdc,min}} = (1 - 0.2 \cdot \alpha_{\mathrm{RA}}) \cdot \frac{11}{\gamma_{\mathrm{V}}} \cdot \sqrt{\frac{f_{\mathrm{ck}}}{f_{\mathrm{yd}}} \frac{d_{\mathrm{dg}}}{d_{\mathrm{v}}}}$ The parameter d_{dg} taking account of concrete type and its aggregate properties shall be assumed as d_{dg} = 16 mm.
Deflection control	$\zeta = 1 - \beta_{\mathrm{tRA}} \cdot \left(\frac{\sigma_{\mathrm{sr}}}{\sigma_{\mathrm{s}}}\right)^2$ where $\beta_{\mathrm{tRA}} = 1.0$ for single, short-term loading $\beta_{\mathrm{tRA}} = 0.25$ for sustained or repeated loading
Concrete cover for durability	Determine exposure resistance by testing if relevant. For concrete including recycled aggregate, the same minimum cover depth for durability $c_{min,dur}$ applies provided the material pertains the same exposure resistance class as concrete including natural aggregate only. If exposure resistance is not determined, for reinforced concrete and for prestressed concrete when $\alpha_{RA} > 0$, the values of $c_{min,dur}$ should be increased by 5 mm for exposure to carbonation and 10 mm for exposure to chloride ingress

ρ_{RAC} – density of RAC; E_{cm} – modulus of elasticity; k_E – factor dependent on the type of NA (can be taken as 9500);
f_{cm} – mean compressive strength of concrete; $\varepsilon_{cs,RAC}$ – shrinkage strain of RAC; ε_{cs} – shrinkage strain of NAC;
φ_{RAC} – creep coefficient of RAC; φ – creep coefficient of NAC; $\tau_{Rd,c}$ – shear stress resistance of members without shear reinforcement;
γ_V –partial factor for shear resistance; ρ_l –longitudinal reinforcement ratio; f_{ck} – characteristic compressive strength of concrete;
d_{dg} – size parameter describing shear failure zone roughness; d_v – shear-resisting effective depth;
$\tau_{Rdc,min}$ – minimum shear stress resistance; ζ – tension stiffening distribution coefficient; σ_{sr} – stress in tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking; σ_s – stress in tension reinforcement;
$c_{min,dur}$ – minimum concrete cover due to durability requirements

Next, the shear resistance of the slabs were calculated according to the expressions in Table 1. According to prEN1992-1-1, γ_V = 1.4 and d_{dg} = 16 mm + $D_{lower} \leq$ 40 mm, where D_{lower} is the "smallest value of D for the coarsest fraction of aggregates in the concrete permitted by the specification of concrete" [11]. In this case, D_{lower} was adopted as 8 mm (considering the smallest coarsest fraction to be 4/8 mm) so that d_{dg} was 24 mm for NAC and—due to the limitation in Table 1—16 mm for RAC. The design strength of reinforcement f_{yd} was obtained as $f_{yk}/1.15$.

Figure 1 presents the ratio between shear resistance ($\tau_{Rd,c} \cdot b \cdot d$) and shear action (calculated as $q_{Ed} \cdot L/2$, where q_{Ed} is the design load), plotted against the L/d ratio.

It can be seen that shear strength is satisfied for all concrete with a significant margin, although there is a notable reduction between NAC and RAC due to the limitation of d_{dg}; the effect of α_{RA} is minor as can be seen from the differences between RAC 0.2 and RAC 0.4. The difference between NAC and RAC is approximately 25% for both concrete classes.

In the second stage deflections a were calculated and compared with permissible deflections $a_{lim} = L/250$. For this purpose, the ζ-method of interpolating deflections was applied:

$$a = \zeta \cdot a_2 + (1 - \zeta) \cdot a_1 \tag{1}$$

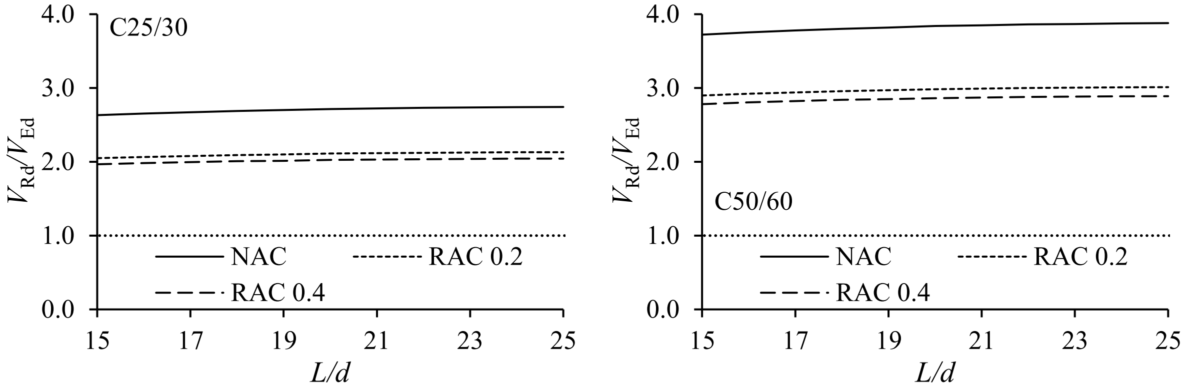

Figure 1. Ratio of shear resistance to shear action vs. L/d for concrete C25/30 (left) and C50/60 (right)

where a_1 and a_2 are deflections in the uncracked and fully-cracked state, respectively. The deflections in states 1 and 2 are composed of a component due to load a_{load} and a component due to shrinkage a_{cs} and are calculated as

$$a_{\mathrm{load,i}} = K \cdot \frac{M \cdot L^2}{E_{\mathrm{c,eff}} \cdot I_{\mathrm{i,i}}} \tag{2}$$

$$a_{\mathrm{cs,i}} = \delta_{\mathrm{cs}} \cdot \varepsilon_{\mathrm{cs}}(t, t_{\mathrm{s}}) \cdot \frac{S_{\mathrm{i,i}} \cdot L^2}{I_{\mathrm{i,i}} \cdot 8} \tag{3}$$

where i can take values 1 or 2; K depends on the statical system (e.g., 5/48 for a simply supported beam under uniformly distributed load); $I_{i,i}$ is the moment of inertia of the transformed section; $S_{i,i}$ is the first moment of area of the reinforcement about the transformed section's centroid; $\varepsilon_{cs}(t,t_s)$ is the concrete shrinkage strain at time t with drying initiation at time t_s; δ_{cs} depends on the statical system (e.g., 1 for a simply supported beam); and the effect of creep is taken into account using the effective modulus of elasticity $E_{c,eff}$:

$$E_{\mathrm{c,eff}} = \frac{1.05 \cdot E_{\mathrm{cm}}}{1 + \varphi(t, t_0)} \tag{4}$$

where $\varphi(t,t_0)$ is the creep coefficient at time t for concrete loaded at time t_0. For simplicity, in this study, a single load application time was adopted for all loads, i.e. t_0 = 28 days. Both shrinkage strain and creep coefficient were first calculated using the prEN1992-1-1 models for NAC and then, for RAC, corrected using adjustments provided in Table 1.

The results are shown in Figure 2 in terms of the a/a_{lim} ratio vs. the L/d ratio. This representation allows easy detection of L/d ratios above which permissible deflections are exceeded (shown by the dotted horizontal line). It should be noted that for all slabs, only the ULS-necessary reinforcement $A_{s,ULS}$ was adopted.

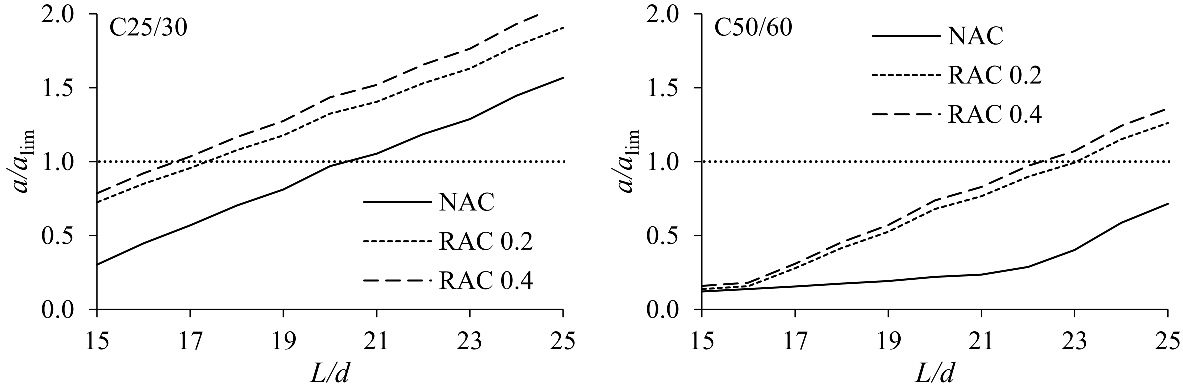

Figure 2. Ratio of deflection to permissible deflection vs. L/d for concrete C25/30 (left) and C50/60 (right)

From the figure, it can easily be seen that the deformability of RAC slabs is predicted as being larger than for NAC slabs, with only minor differences between RAC slabs themselves. Namely, for concrete C25/30 the lines for all three concretes stay parallel, with a constant "absolute" difference ($(a/a_{lim})_{RAC} - (a/a_{lim})_{NAC}$) of the a/a_{lim} ratio between RAC and NAC: for RAC 0.2 this difference is 0.34–0.42 and for RAC 0.4 it is 0.46–0.50. The figure also demonstrates that NAC slabs can satisfy deflections up to L/d = 20, whereas this limit is around 16.5–17.0 for RAC 0.2 and 0.4.

For C50/60, because of its higher tensile strength, all slabs remain uncracked for certain L/d ratios (until 22 for NAC and until 16 for RAC). The earlier cracking of RAC is due to the decreased tension stiffening effect (Table 1). From L/d = 22, the lines for RAC and NAC become parallel with an absolute difference of 0.55–0.60 for RAC 0.2 and 0.60–0.70 for RAC 0.4. Finally, it can be seen that NAC can satisfy deflections for all considered L/d values, whereas for RAC, the limit is at approximately L/d = 22.5–23.0.

Although the deformability of one-way reinforced RAC slabs seems significantly larger than that of NAC slabs, it should be noted that the results demonstrate their applicability in building structures. Namely, even for a very low concrete strength class C25/30 and using only the ULS-required reinforcement, one-way RAC slabs satisfy deflections up to L/d = 17 and for C50/60 this increases up to 22.

3.3 Strategies for compliance with deflection limits for RAC and NAC one-way slabs

In order to further analyze and quantify the differences in RAC and NAC one-way slab behavior, two strategies were investigated in this section. The objective was to determine necessary changes to RAC and NAC one-way slabs in order for them to comply with deflection limits (i.e., achieving $a/a_{lim} \leq 1.0$).

First, it was investigated how much longitudinal tensile reinforcement needs to be increased in order for each slab to comply with deflection limits. If the original value of the a/a_{lim} ratio was already below 1, no changes were needed. The necessary multiplication factors for longitudinal reinforcement are presented in Table 2. For certain cases, deflection compliance cannot be achieved by increasing reinforcement and these cases are marked as "n/a".

Table 2. Necessary increases in tensile reinforcement ($A_{s,prov}/A_{s,ULS}$) in order to achieve $a/a_{lim} \leq 1.0$.

L/d	C25/30			C50/60		
	NAC	RAC 0.2	RAC 0.4	NAC	RAC 0.2	RAC 0.4
15	1.00	1.00	1.00	1.00	1.00	1.00
16	1.00	1.00	1.00	1.00	1.00	1.00
17	1.00	1.00	1.10	1.00	1.00	1.00
18	1.00	**1.22**	**1.65**	1.00	1.00	1.00
19	1.00	**1.61**	***2.58***	1.00	1.00	1.00
20	1.00	***2.69***	***10.39***	1.00	1.00	1.00
21	**1.16**	***3.93***	***n/a***	1.00	1.00	1.00
22	**1.71**	***n/a***	***n/a***	1.00	1.00	1.00
23	***2.48***	***n/a***	***n/a***	1.00	1.00	**1.23**
24	***7.82***	***n/a***	***n/a***	1.00	**1.48**	***2.22***
25	***n/a***	***n/a***	***n/a***	1.00	***2.07***	***n/a***

It can be considered that increases of reinforcement up to 100% (i.e. a multiplication factor of 2.0) are acceptable in these cases as these slabs have very low reinforcement ratios when $A_{s,ULS}$ is considered (0.15%–0.30%). In this way, the use of RAC slabs can be extended up to L/d of 19 and 18 for RAC 0.2 and RAC 0.4, respectively, for C25/30 and up to L/d of 24 and 23, respectively, for C50/60.

Another option for complying with deflections is the increase of slab thickness, preferably in combination with increasing reinforcement. For this purpose, all cases from Table 2 that could not comply with deflections with an increase in reinforcement of 100% (given in italics in Table 2), were limited to $A_{s,prov}/A_{s,ULS}$ = 2.0, after which effective depth d was increased until deflection limits were satisfied.

For C25/30 and NAC, the cases with L/d > 23 required a slab effective depth of 275 mm. For C25/30 and RAC 0.2 and the cases with L/d > 20, d = 315 mm was required, whereas for C25/30 and RAC 0.4 with L/d > 19, d = 330 mm was required. For NAC and C50/60, no increase in d was necessary, for the same concrete class and RAC 0.2 with L/d = 25, d = 245 mm was needed and for RAC 0.4 and L/d > 24, d = 255 mm. Finally, the required ratios d_{RAC}/d_{NAC} for achieving compliance with deflections are shown in Table 3. Table 3. Increases in RAC slab effective depth relative to NAC (d_{RAC}/d_{NAC}) in order to achieve $a/a_{lim} \leq 1.0$.

It can be seen from the table that for the lowest concrete class C25/30, the necessary increase in effective depth is limited to 22% for RAC 0.4, whereas this is only 6% for C50/60. Therefore, it can be concluded that when using RAC slabs, higher strength classes should be aimed for, as well as lower RA substitution ratios and applications requiring lower L/d ratios.

Table 3. Increases in RAC slab effective depth relative to NAC (d_{RAC}/d_{NAC}) in order to achieve $a/a_{lim} \leq 1.0$.

L/d	C25/30		C50/60	
	RAC 0.2	RAC 0.4	RAC 0.2	RAC 0.4
15	1.00	1.00	1.00	1.00
16	1.00	1.00	1.00	1.00
17	1.00	1.00[a]	1.00	1.00
18	1.00[a]	1.00[a]	1.00	1.00
19	1.00[a]	1.03[b]	1.00	1.00
20	1.05[b]	1.10[b]	1.00	1.00
21	1.09[b]	1.14[b]	1.00	1.00
22	1.15[b]	1.20[b]	1.00	1.00
23	1.15[b]	1.22[b]	1.00	1.00[a]
24	1.15[b]	1.22[b]	1.00[a]	1.02[b]
25	1.15[b]	1.22[b]	1.02[b]	1.06[b]

[a]deflections satisfied only with increased reinforcement ($A_{s,prov}/A_{s,ULS} < 2.0$)
[b] $A_{s,prov}/A_{s,ULS} = 2.0$

4 Conclusions

This study presents the new provisions of EC2 for the design of RAC structures, as well as the study of the implications of these provisions on the design of one-way slabs. For this purpose, a parametric study of RAC one-way slabs is performed with ULS and SLS design carried out. Additionally, the higher deformability of RAC one-way slabs in terms of deflections is discussed together with strategies for satisfying deflection criteria. Based on the findings of the study, the following can be concluded:

- The provisions of Annex N of the new EC2 provide a detailed framework for the structural design of RAC members, considering differences in both material and structural properties between RAC and NAC.
- The ULS design of RAC simply-supported one-way slabs does not provide any substantial differences compared with NAC design. The only difference exists in shear resistance calculation; however, for the range of *L/d* values considered in this study, and the adopted loading, there are no implications of these differences.
- Significant differences exist in the deformability (i.e., deflection behavior) between RAC and NAC one-way simply supported slabs, particularly for the low concrete strength class C25/30. Nonetheless, RAC slabs remain applicable in the typical slenderness ranges for this type of element.
- Increasing reinforcement above the ULS-necessary amount can in a limited number of cases lead to satisfying deflection limits. However, in the majority of cases, this measure needs to be combined with an increase in effective depth. For concrete class C25/30, the increase of RAC effective depth, relative to NAC can be up to 22%, whereas it is only 6% for C50/60. Therefore, RAC one-way slabs should be used in higher strength classes whenever possible, as well as with lower RA substitution ratios ($\alpha_{RA} = 0.2$).

While indicative, the conclusions of this study are not conclusive and cannot be extrapolated beyond the range of parameter values adopted herein. Further and more detailed studies are needed to cover a wider range of statical systems, member types, loading arrangements, etc. Nonetheless, the study provides a first step towards familiarizing designers with the new provisions of EC2 for the structural design of RAC.

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Short project notes

Construction of the shopping center Ada Mall in Belgrade

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steel structure

ABSTRACT
Ada Shopping Mall is located at the corner of Radnička and Paštrovićeva Street in Belgrade, closed to Lake Ada Ciganlija. The building has a gross area of about 100.000m². It consisted of reinforced concrete structure with three underground and four above-ground levels and a steel roof structure. The foundation pit protection was made by bored and bored secant piles in combination with two different Top-Down methods during construction. The excavation depth was up to 25 m. As the building was built in rock mass, during excavation blasting was applied. During construction a monitoring system was implemented. This paper provides an overview of the construction process of shopping mall.

1 Introduction

In May this year it will be exactly two years since the completion of construction of the shopping center Ada Mall, located at the corner of the Radnička and Paštrovićeva street in Belgrade, close to lake Ada Ciganlija. In this article it will be explained how the building was constructed, which construction technologies were applied and what kind of problems constructors had to solve.

Figure 1. Shoping Center Ada Mall under construction

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2 Reinforced concrete structure

The structure was of irregular basis, which should simulate the isohypses of the hill that was cut down and the future shopping center was inserted in that place.

The building with a gross area of about 100,000 m² consists of reinforced concrete structure with three underground and four above-ground floors and a steel roof structure. The spans between the columns are mostly 8x8m, while their height was up to 26 m. The floor heights of the underground floors are 3.20 m and 3.90 m, and the above-ground 5.5 m and 6 m.

The Floor slabs were geometrically and performingly very complex and challenging. Flat slab was supported directly by columns with exception in zones with large spans where high beams or post tensioned beams were introduced. The architectural requirements were such that it was not always possible to keep the continuity of the columns, so this problem was solved by transfer beams. The construction abounds in many cantilevers, denivelations and a large atrium opening in the middle with two footbridges. The Floor slabs have thickness of 25cm and 30 cm, and the gross area of one Floor slab is up to 16,000 m².

3 Foundation pit protection, top-down method and construction of the building

As already mentioned, the building was cut into the existing hill, and additional to that, there was a request for a large number of parking places, so three underground floors were designed. According to these requests, the excavation depth along Radnička Street was 12 m and almost 25 m along Visoka Street.

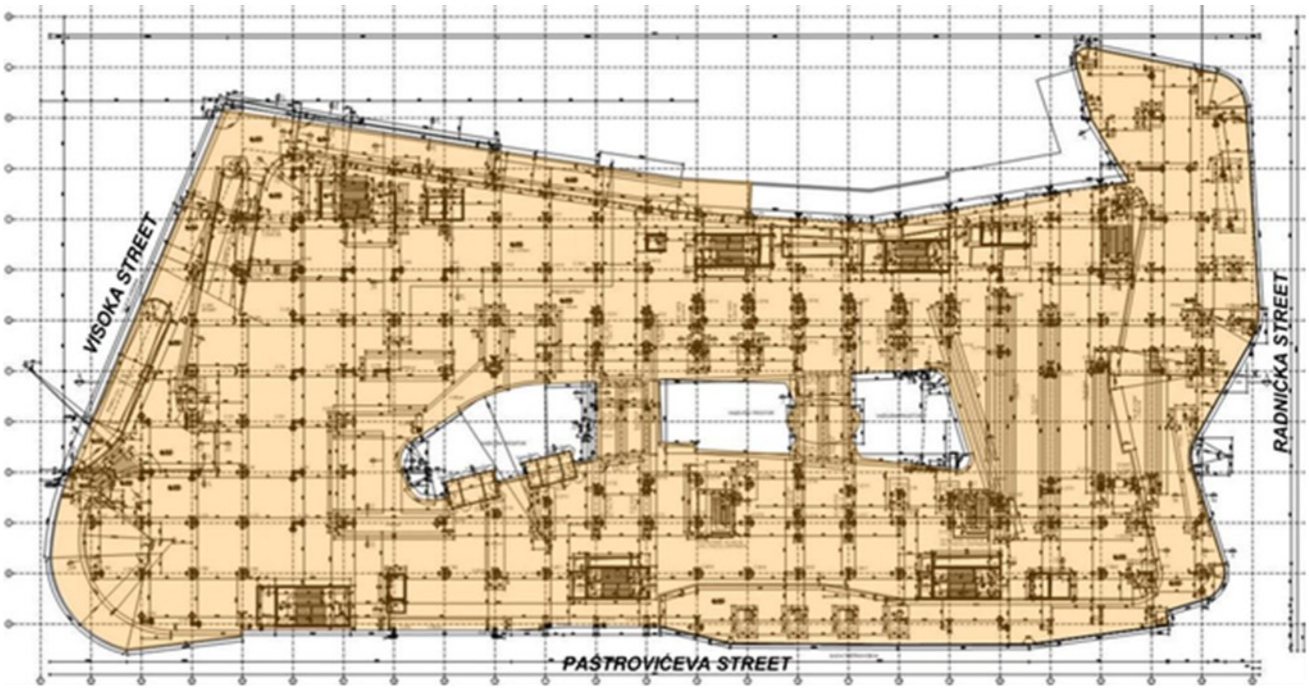

Figure 2. The Third Floor Slab Plan

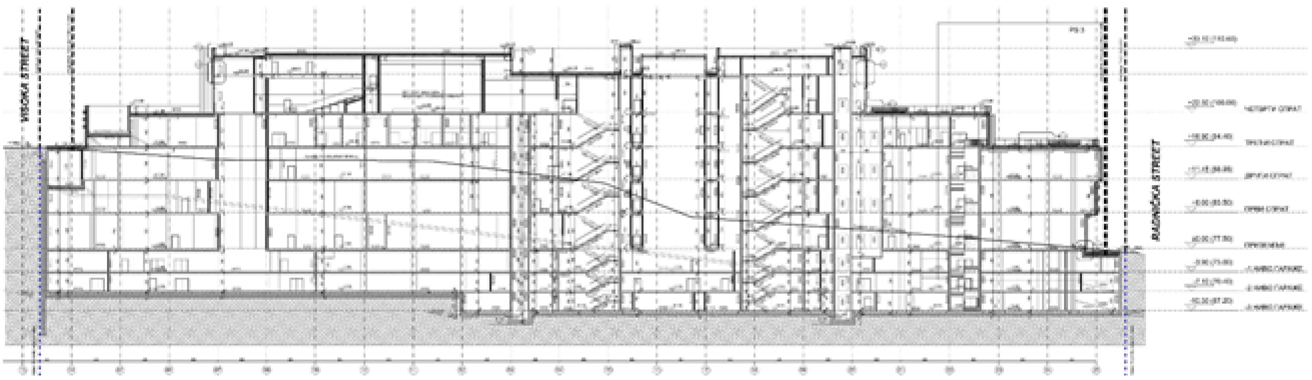

Figure 3. Longitudinal section of the building in the direction from Visoka Street (left) to Radnička Street (right)

The protection of the foundation pit and excavation was done with drilled piles along the perimeter in combination with the construction of the building in the top-down method. The perimeter wall was strutted by floor slabs, and their construction followed the excavation.

Accordingly, the building was divided into Part A (lower terrain - along the lake and Radnička Street) and Part B (higher terrain - along Visoka Street).

Figure 4. Construction Plan - division of the building into part A and B and application of two types of top-down methods 1 and 2

Figure 5. Construction Plan - division of the building into parts A and B and application of two types of top-down methods 1 and 2

3.1 Monitoring

Before the construction started, a monitoring system was set up to control the entire construction of the building 24 hours a day:

– displacement on inclinometers installed within perimeter piles
– rotation on tiltmeters
– vibrations (due to blasting and construction of the building)
– settlement and displacement of neighbour buildings by geometric observation
– noise.

Thus, it was easy to detect and follow any suspicion during construction. With the help of inclinometers and tiltmeters, information was obtained on whether there was displacement of neighbouring buildings, neighbouring roads and the construction of the foundation pit protection.

All this information was posted on the website from where the required result could be read at any time, and the telephone of managers of the project were directly connected to the server so that, in case of any displacements, they would receive a text message which would warn them of the problem and they could take necessary action after working hours as well.

3.2 Construction of part a of the building

Part of the building along the lake had problems with groundwater because of the proximity of the Sava River. Therefore, secant piles were chosen for pit protection, and they were drilled up to depth of waterproof layers of the soil. Horizontal drainage was used as a permanent solution for groundwater and high buoyancy. It consists of the system of channels with a collecting pool from which water will be permanently pumped out during the lifetime of the building.

Figure 6. Reading data from inclinometer within pile

Figure 7. Noice recording

Figure 8. Tiltmeter

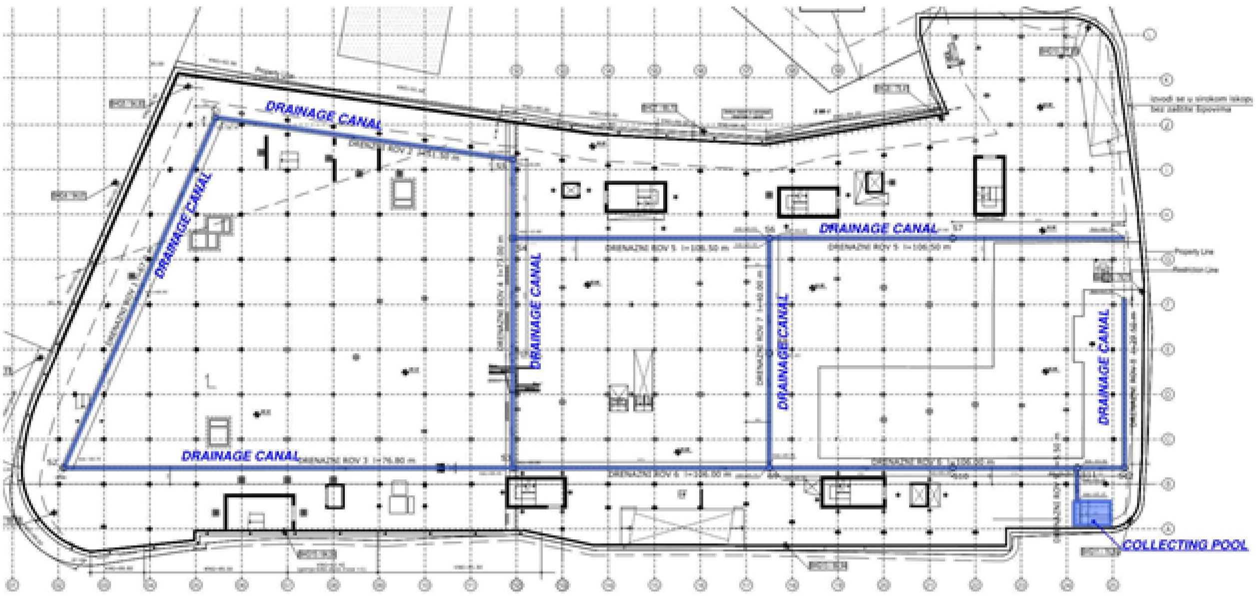

Figure 9. Horizontal drainages channel system with large collecting pit (between axes 24-25 / 01-02)

After the secant piles along the perimeter and the piles at the column locations in the berm zone were completed, wide excavation started up to the bottom of the foundation slabs in part A with the formation of the berm, which can be seen in Figures 10 and 11.

Figure 10. Part A wide excavation (up to the depth of 12 m) with the formation of a berm (zone 1) where piles were drilled at the column locations

Figure 11. Part A wide excavation with formed berm (zone 1) where piles were drilled at the column locations

Since the steel piles were made at the column locations, the top of the berm was levelled, and a formwork was installed on the ground, which will remain trapped until the excavation of the berm starts. See Figure 12.

Construction of the building started from the foundation slab up to the height of the first-floor slab from where the berm was bridged. Head beam above secant piles was strutted by floor slab. In places where the span of the slab was large, temporary steel columns (piles) and / or a temporary steel frame structure were previously made (before excavation).

Figure 12. Installed formwork at the surface of the berm

Figure 13. Part A under construction

Figure 14. Strutting by -1B slab over the berm (the slab is supported by temporary steel frame and steel pile (future RC columns)

Thus, it was possible for the building to emerge from the ground very quickly and continue its construction unhindered, while the berm below was removed paralely. Excavated soil was exported from the building. As there was flysch in this part of the building, the berm was excavated with the help of powerful hammers and rippers that were mounted on excavators (Figure 15).

Figure 15. Excavation of the berm under finished structure with hydraulic hammers and rippers

Figure 16. Excavation of the berm under finished structure. Secant piles, temporary steel columns (piles), formwork installed at the berm

Excavation was followed by careful uninstalling of the formwork.The steel columns were secured with struts before they were completely released, depending on their current load, and then concreted to the height of the first next floor.

Extremely strong rock mass (Figure 5.) appeared in one part of construction site, near river, whose strength was up to 190 MPa of axial stiffness. It was impossible to do excavation in this zone. Machines broke down, hammer spikes cracked, diamond teeth on rangers of pile drilling machines melted.

Nothing else could be done except blasting. Considering that the construction site was in the urban part of the city, blasting was neither simple nor easy. Neighbouring residential buildings had to be considered. Additional devices were installed on neighbouring buildings in the blasting zone, and they measured the displacement (horizontal and vertical) whose results had to be within the permitted limits.

Figures 17-21. Broken drilling rigs, diamond teeth on the ranger and the spikes of a hydraulic hammer

Figure 22. Drilling a hole for explosives

The blasting itself was done with nonel (explosive) with double decelerators so that larger number of explosions could go one after the other with a delay of a few milliseconds. Its complexity clearly shows that it was a special blasting, which was in many ways different from that in quarries outside the urban environment.

The following picture shows the area along Radnička Street that was blasted. In this zone vertical excavation was done without protection.

Figure 23. The explosive was inserted into the holes and interconnected with decelerators

Figure 24. Extremely strong rock zone where vertical excavation was done without protection

There was a huge problem with the stability of the berm along Paštrovićeva Street, at the place where the transformer station of the factory and the old warehouse used to be. Oil leaked at that place for years, so the quality of the soil was weakened. The ground collapsed during the excavation and there was a danger that the stability of the street would be endangered.

The steel frame structure between the berm and the building could not be mounted due to the danger of landslides. On the other hand, even the formwork on the ground could not be installed, because the berm itself could not hold the load of the new slab.

Strutting by steel pipes was applied as one of the possible solutions. See Figure 25.

Figure 25. Strutting of the building in part A, zone along Paštrovićeva Street

Figure 26. Part A of the building under construction in an advanced phase

3.3 Constructionof part B of the building

This part of the building was done on a much higher terrain; the depth of the excavation was up to 25 m. Therefore, a different construction technology was applied in relation to the part A.

Drilled piles Ø800 and Ø1000mm were made along the perimeter of the structure, as part of the protective structure of the excavation. Construction began at the level of the third floor (Top-Down zone 2). The hill was levelled so that the pile drilling machines could move. Steel piles over 23 m long were made on the places of future columns and walls. Later, in the top-down phase, these piles are supports on which the entire structure stands, while one floor after another is being excavated successively under the built object.

Figure 27. Construction of part B, zone 2

Figure 28. Construction of part B, Top Down 2

Due to install the steel pipe within the dimensions of the future column or wall, the piles are very long from the point of required high precision. Any deviation from the verticality and the projected position means additional problems in the top-down where the steel pipe has to be cut and shortened to provide correct position according to the project and not to enter the parking space or garage area.

A special concrete platform was provided for this project. Concrete platform contained a hole in the middle which was aligned with the drilled hole for pile. On the top of the platform a steel pipe with a "cross" is passed through the mentioned hole, which is a guideline. The "cross" had the possibility of fine adjusting at its ends. Precisely, that pipe-guide shapes the direction of the installation of steel pile-column. See Figure 29. Figure 30 shows a steel pile instalation. After that, the reinforcement cage is installed, and then the pile is concreted. See Figure 32.

Figure 29. Concrete platform with steel guide

Figures 30 and 31. Installation of the steel pile through the “guide”for pile installation

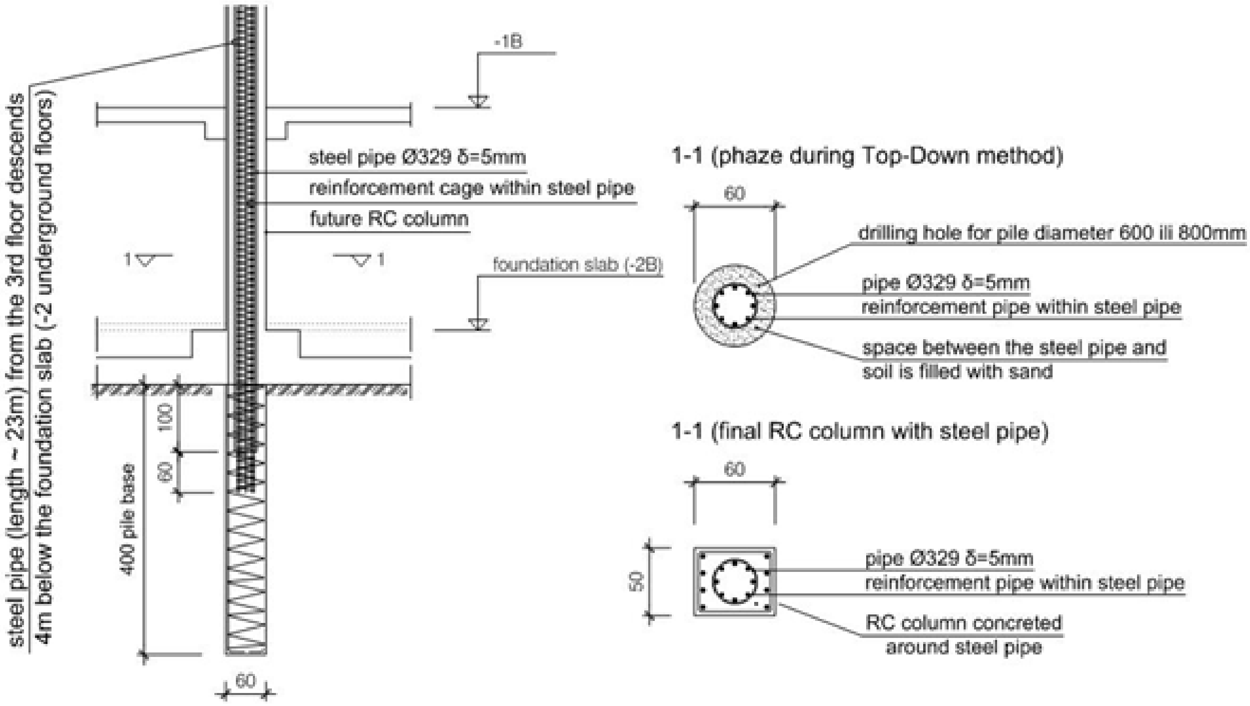

Figure 32. Typical pile at the column location

Figure 32. shows a typical pile at the column location. A hole with a diameter of 600 mm or 800 mm is drilled at column location from the third floor to a depth of 4 m below the bottom of the foundation slab (through 5 floors, it is slightly more than 23 m). A reinforcing cage for the pile base is put down through mentioned hole and concrete is poured right after. A steel pipe with a diameter of 329 mm and the reinforcement of the pile are dropped into the fresh concrete. The next day, the steel pipe was filled with concrete, and the hole was filled with sand on the outside (between steel pipe and soil). Sand is very important because of a buckling prevention for the pipe (steel column) when the load from the upper floors is applied. Additionally, it is easy to release the sand around the pipe and clean steel surface to provide welded connection between the steel pipe and slab.

Since all the piles were built at the column locations of zone 2, the formwork of the third-floor slab was placed on the ground. This formwork remained captured until the excavation under began.

The slab formwork consisted of the formwork tables with dimensions 4mx4m which remained attached to the concrete slab also at the moment of excavation in the top-down phase. Later the same tables were lowered with the help of straps and birches (see Figures 41 and 42).

The reinforcement of the slab was done over the formwork tables. Steel structure, which enabled the connection between the piles and the slab, was welded to the pile in the zone of steel piles - future columns. See Figure 34.

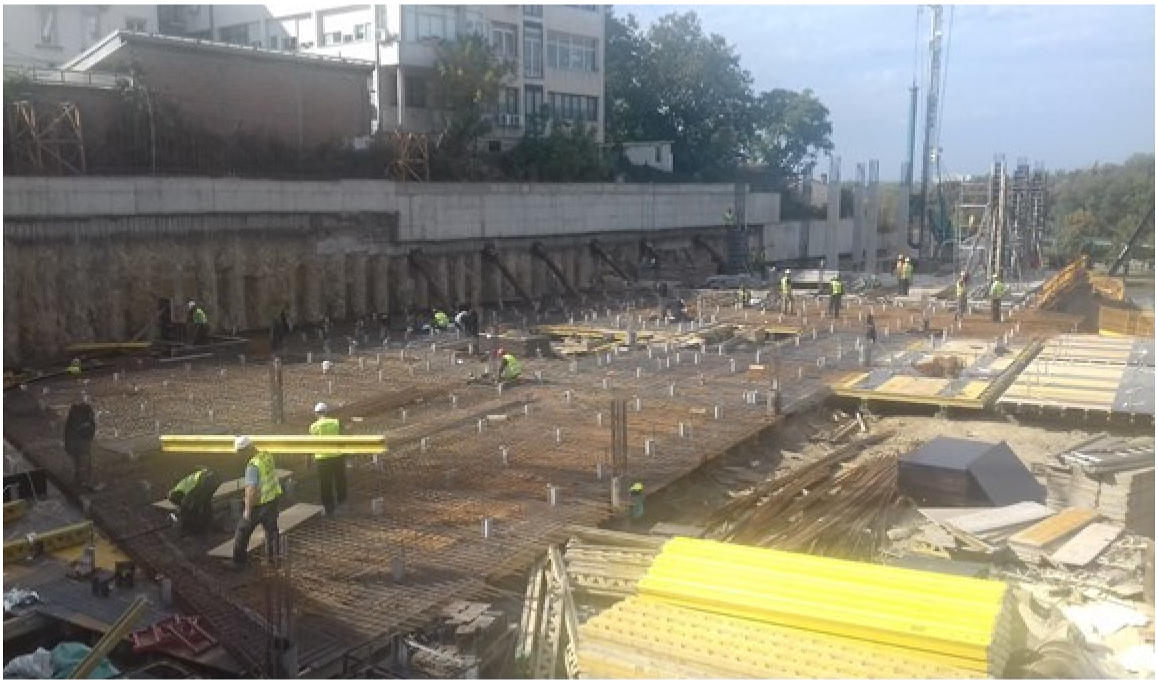

Figure 33. Installation of formwork tables of the third-floor slab on the ground (Part B, zone 2)

Figure 34. Steel structure for connection of piles and slab

After concreting the slab of the third floor, the construction of the next floor continued upwards.

Due to the slope of the terrain, the top down 2 (as indicated in Figures 4 and 5) of the part B of the building, could not be done before merging part A and part B of the building.

Figure 35 .Part A is in an advanced phase and is progressing towards part B where the construction started from the 3rd floor upwards

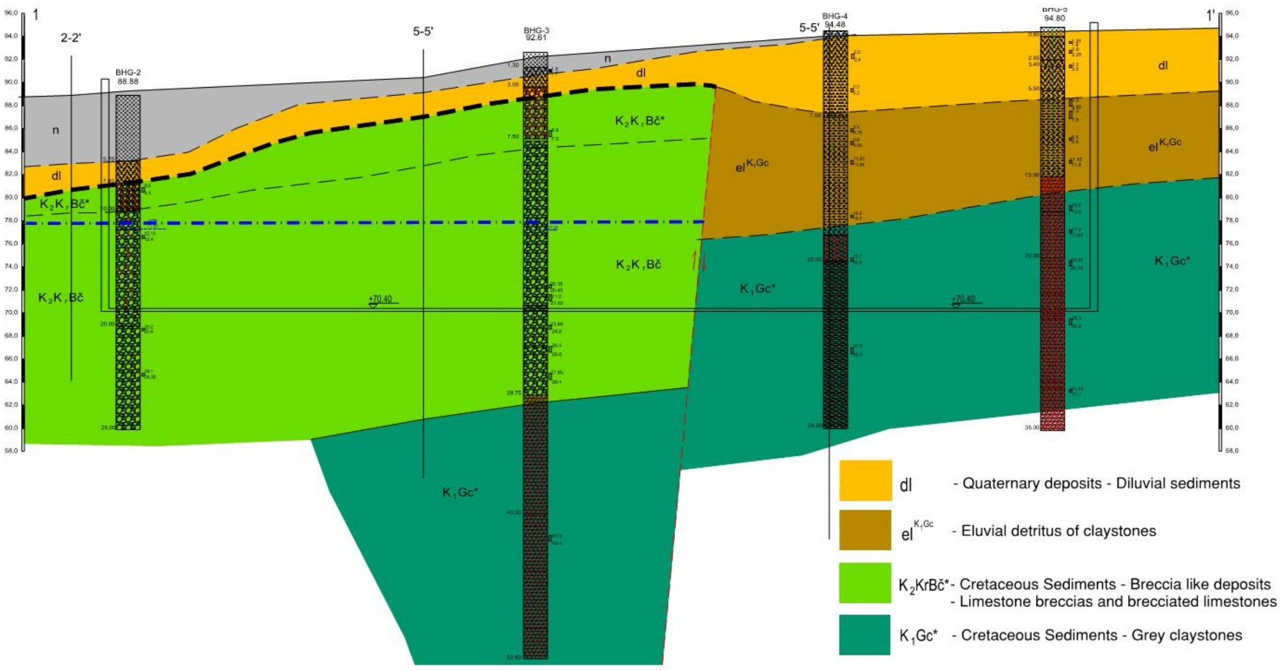

Figure 36. Fault in zone 2 and transition from limestone breccias to clays

The main problem was the neighbouring building which provided large horizontal forces on structure under construction. As zone 2 is the only one with previously erected building (from the third floor upwards), it was necessary to finish the whole zone 1 of part B (Figures 4 and 5) in order to start the top down works below that part. This enabled the horizontal forces to transfer from one side to the other, i.e. to provide the existing structure to be strutted by head beam and the perimeter piles along Paštrovićeva Street. The fault and transition from the limestone breccias to claystones was additional problem under the existing structure (zone 2).

As we mentioned, after merging the parts A and B the conditions for starting the top-down method in zone 2 and building the floors down (from the third floor to the level 2) are met.

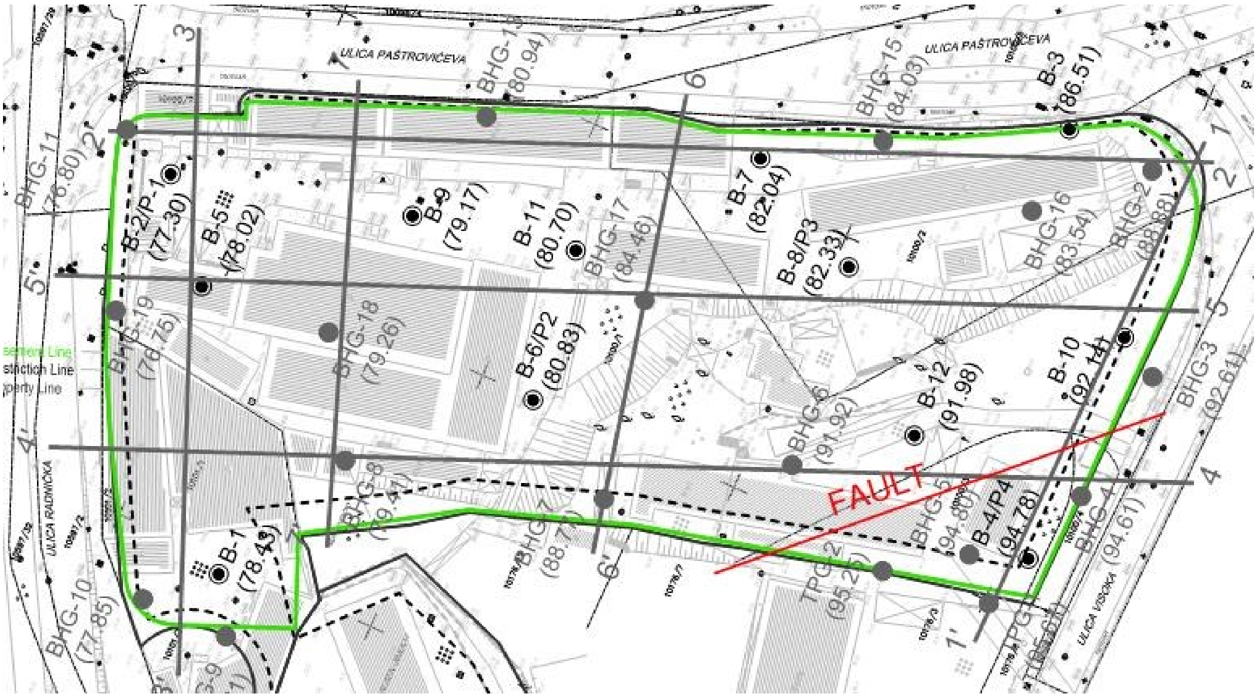

Figure 37. The direction of the fault in zone 2 and the transition from limestone breccias to clays

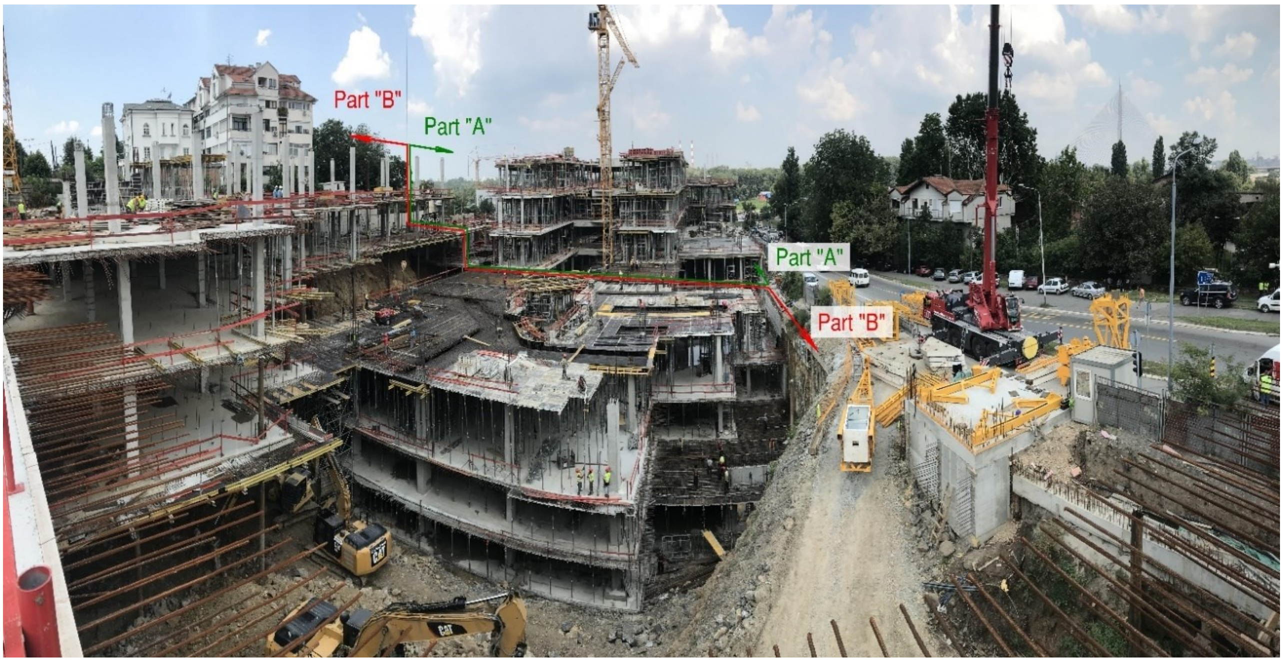

Figure 38. The first connection of the parts A and B was made with the third floor slab

The excavation under the formwork tables began. It is important to emphasize that in zone 2 as well as in the entire zone B and most of the zone A, excavation is constantly difficult due to the rock mass that had to be blasted using the already mentioned technique.

Figure 39. Blasting under formwork tables and the third floor slab (part B, zone 2)

Figure 40. Excavation in the top-down phase below the constructed part of the building (part B, zone 2)

After finishing excavation under the existing slab at the height of one floor, the formwork tables are lifted down to the next lower level.

Figure 42. shows the anchors for the column that are left from the upper floor for the column below, which will be done in the next phase. It is the same with the walls. Both the pillars and the walls are concreted with self-compacting concrete from the upper floor through previously left holes in the slab.

Figure 42. One of the hunged slab in the top-down phase (part B, zone 2)

Figure 43. Shutter erection of formwork of the perimeter wall - formwork with one-sided formwork

Figure 44. Column reinforcement installing around the steel pipe of the pile

After the excavation reached the foundation slab, in the specific case of five underground floors in the top down work, the complete mechanization passes through the entire building and exits through the prepared ramp in the zone towards Radnička Street.

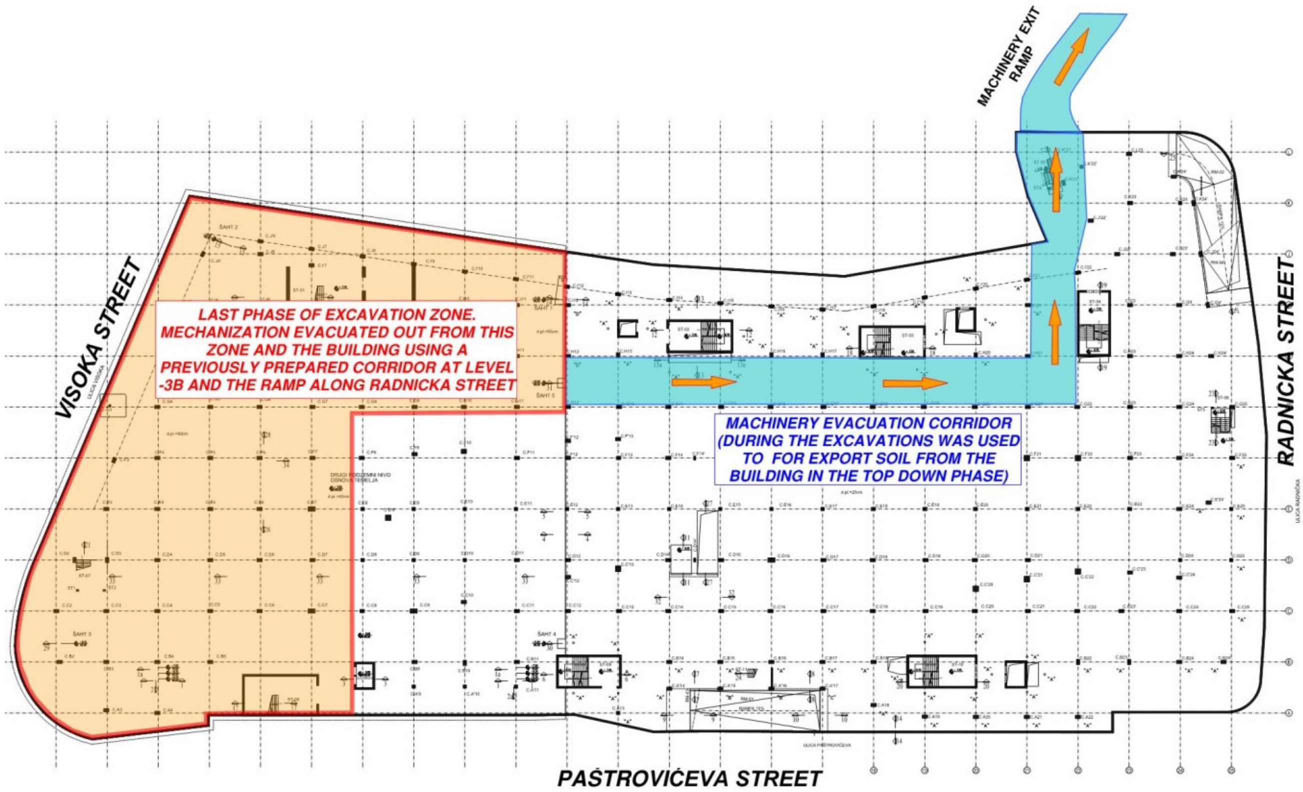

Figure 45. Scheme of the last phase of excavation and evacuation of machinery from the building

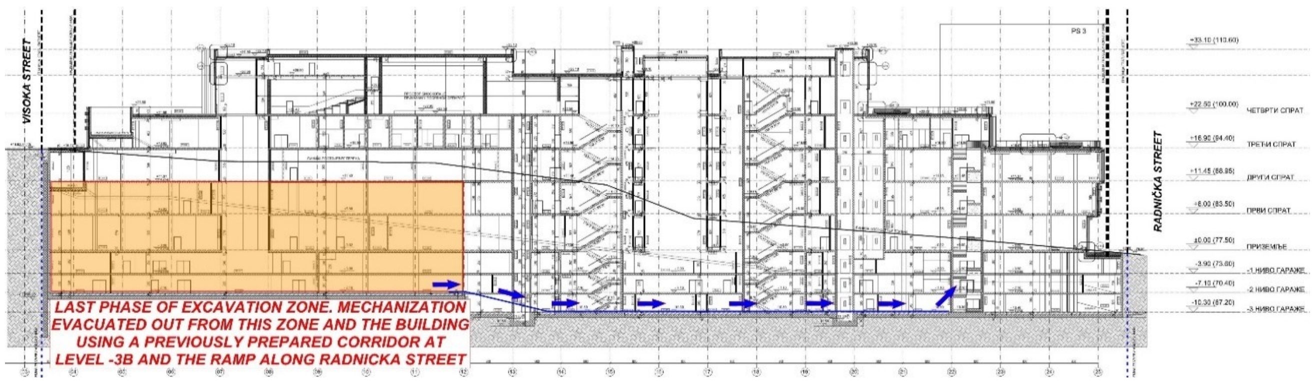

Figure 46. Scheme of the last phase of excavation and evacuation of machinery from the building

Figure 47. Cancellation of the temporary exit ramp

Figure 48. Temporary ramp is cancelled. Mechanization from Visoka Street passes under the building to Radnicka street going outside

Figure 49. Passage of machinery under the building and exit to the ramp along Radnička Street

3.4 Construction of the last floor of the entire building and steel roof structure

After merging the parts A and B of the building, regardless of the top-down works, the last floor could be completed.

Construction of the roof steel structure started after completition of construction of concrete structure above ground. As the surrounding streets were extremely busy, installation of steel structure was performed from the atrium zone of the ground floor slab, and inaccessible parts were finished with the help of cranes that remained inside the building.

Figure 50. Unobstructed construction of the last floors while the top down works are performed in the underground

Figure 51. Installation of the steel roof structure from the atrium zone of the ground floor slab

Figure 52. Steel structure under construction

Figure 53. The construction of steel and concrete structure in parallel

Figure 54. Anchor plate for steel column

4 Problems

There were other problems in addition to the mentioned ones regarding blasting, the high strength of the soil and the top-down method.

Extremely complicated geological structure of the soil and the frequent transitions from one type of soil to another, which was not predicted by Geomechanical Report, provided a new problem for construction. There was a septic tank for several houses on the adjacent plot. Formed berm was constantly soaked by it, and numerous layers of bad material between the two rock masses led to the formation of a sliding plane which caused damage of the steel columns in local area

Figure 55. Different layer of the bad soil in the rock mass

Figures 56 and 57. Damaged steel columns due to a local landslide

Figures 58 and 59. The failure of the steel column after the landslide. Steel column is bridged by a lattice steel structure placed on the slab above it. The load from the damaged column was transferred to the adjacent columns

In part B of the building, phase 2 of the Top-Down method, a bomb from the Second World War was found between the first and the second floor slabs.That stopped further works for a while.

Figure 60. The place where the grenade was found(between the slabs of the first and the second floor)

Figure 61. World War 2 grenade

5 Conclusion

It should be noted that 43,000 m^3 of concrete, 7,050 tons of reinforcement and 770 tons of roof steel were used for the construction of the Ada Mall. Moreover, 13 tons of explosives were used in order to excavate over 150,000 m^3 of rock out of a total of 250,000 m^3, mostly thanks to the indisputable skills of domestic builders.

INSTRUCTIONS FOR AUTHORS

Acceptance and types of contributions

The Building Materials and Structures journal will publish unpublished papers, articles and conference reports with modifications in the field of Civil Engineering and similar areas (Geodesy and Architecture).The following types of contributions will be published: original scientific papers, preliminary reports, review papers, professional papers, objects describe / presentations and experiences (case studies), as well as discussions on published papers.

Original scientific paper is the primary source of scientific information and new ideas and insights as a result of original research using appropriate scientific methods. The achieved results are presented briefly, but in a way to enable proficient readers to assess the results of experi-mental or theoretical numerical analyses, so that the research can be repeated and yield with the same or results within the limits of tolerable deviations, as stated in the paper.

Preliminary report contains the first short notifications on the results of research but without detailed explanation, i.e. it is shorter than the original scientific paper.

Review paper is a scientific work that presents the state of science in a particular area as a result of analysis, review and comments, and conclusions of published papers, on which the necessary data are presented clearly and critically, including the own papers. Any reference units used in the analysis of the topic are indicated, as well as papers that may contribute to the results of further research. If the reference data are methodically systematized, but not analyzed and discussed, such review papers are classified as technical papers.

Technical paper is a useful contribution which outlines the known insights that contribute to the dissemination of knowledge and adaptation of the results of original research to the needs of theory and practice.

Other contributions are presentations of objects, i.e. their structures and experiences (examples) in the construction and application of various materials (case studies).

In order to speed up the acceptance of papers for publication, authors need to take into account the Instructions for the preparation of papers which can be found in the text below.

Instructions for writing manuscripts

The manuscript should be typed one-sided on A-4 sheets with margins of 31 mm (top and bottom) and 20 mm (left and right) in Word, font Arial 12 pt. The entire paper should be submitted also in electronic format to e-mail address provided here, or on CD. The author is obliged to keep one copy of the manuscript.

As of issue 1/2010, in line with the decision of the Management Board of the Society and the Board of Editors, papers with positive reviews, accepted for publication, will be published in Serbian and English, and in English for foreign authors (except for authors coming from the Serbian and Croatian speaking area).

Each page should be numbered, and the optimal length of the paper in one language is about 16 pages (30.000 characters) including pictures, images, tables and references. Larger scale works require the approval of the Board of Editors.

The title should describe the content of the paper using a few words (preferably eight, and up to eleven). Abbreviations and formulas should be omitted in the title. The name and surname of the author should be provided after the title of the paper, while authors' title and position, as well as affiliation in the footnote. The author should provide his/her phone number, e-mail address and mailing address.

The abstract (summary) of about 150-250 words in Serbian and English should be followed by key words (up to seven). This is a concise presentation of the entire article and provides the readers with insight into the essential elements of the paper.

The manuscript is divided into chapters and subchapters, which are hierarchically numbered with Arabic numerals. The paper consists of introduction and content with results, analysis and conclusions. The paper ends with the list of references. All dimensional units must be presented in international SI measurement units. The formulas and equations should be written carefully taking into account the indexes and exponents. Symbols in formulas should be defined in the order they appear, or alternatively, symbols may be explained in a specific list in the appendix. Illustrations (tables, charts, diagrams and photos) should be in black and white, in a format that enables them to remain clear and legible when downscaled for printing: one to two columns (8 cm or 16.5 cm) in height, and maximum of 24.5 cm high, i.e. the size of the letters and numbers should be at least 1.5 mm. Original drawings should be of high quality and fully prepared for copying. They also can be high-quality, sharp and contrasting photo¬copies. Photos should be in black and white, on quality paper with sharp contours, which enable clear reproduction.

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Authors are responsible for the content presented and must themselves provide any necessary consent for specific information and illustrations used in the work to be published.

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Institut za ispitivanje materijala IMS, sa tradicijom od 1929. godine, predstavlja najstariju naučno-istraživačku instituciju u Srbiji. Osnovna ideja prilikom osnivanja bila je potreba za jedinstvenom institucijom koja bi se osim istraživanja bavila i kontrolom građevinske industrije.

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Izvođenje istražnih radova sa pontona za novi most Beška,2007.god.

Geotehnička istraživanja i ispitivanja – in situ

Od terenskih istražnih radova izdvajamo izvođenje istražnih bušotina (IB), standardnih penetracionih opita (SPT), statičkih penetracionih opita (CPT i CPTU), opita dilatometarskom sondom (DMT i SDMT), ispitivanja vodopropustljivosti tla različitim terenskim metodama (VDP), ugradnja pijezometara i dr.
Terenske metode ispitivanja šipova zauzimaju značajno mesto u našoj delatnosti, a na tržištu se izdvajamo kao lideri u toj oblasti u protekloj deceniji.

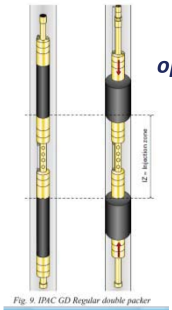

DLT-dinamičko ispitivanje šipova

Ispitivanje šipova

SLT metoda (Static load test) ispitivanje nosivosti šipova statičkim opterećenjem;
DLT metoda (Dynamic load test) ispitivanje nosivosti šipova dinamičkim opterećenjem;
PDA metoda (Pile driving analysis) omogućava praćenje i optimizaciju procesa pobijanja prefabrikovanih betonskih i čeličnih šipova u tlo;
PIT (SIT) metoda (Pile(Sonic) integrity testing) koristi se za ispitivanje integriteta izvedenih šipova (dužine, prekida, suženja ili proširenja).


CPT/CPTU opiti


Aktivno klizište



oprema za ispitivanje vodopropusnosti stena pod pritiskom do 10 bar-a metodom LIŽONA

Laboratorija za puteve i geotehniku

Laboratorija za puteve i geotehniku akreditovana je kod Akreditacionog tela Srbije – ATS prema SRPS ISO/IEC 17025:2006. U njoj se vrše ispitivanja tla (identifikaciono–klasifikaciona ispitivanja, fizičko–mehanička modelska ispitivanja), kamenog agregata i brašna, bitumena i bitumenskih emulzija, asfaltnih mešavina. U okviru laboratorijskih ispitivanja na terenu vrši se kontrola kvaliteta ugrađenog materijala i izvedenih radova (prethodna, tekuća, kontrolna ispitivanja i izvođenja opita in situ).

Projektovanje puteva i sanacija klizišta

U okviru projektovanja značajno mesto u radu zauzimaju geotehnička istraživanja terena i projekti sanacije klizišta - nestabilnih kosina useka i nasipa puteva i prirodno nestabilnih padina . Značajna su i projekovanja svih vrsta fundiranja specijalnih geotehničkih konstrukcija. Ističe se i iskustvo u oblasti putarstva, na projektovanju novih, rehabilitacija i rekonstrukcija postojećih puteva svih rangova sa pratećim objektima i dimenzionisanjem kolovoznih konstrukcija.

Nadzor

Naši inženjeri imaju veliko iskustvo u kontroli i proveri kvaliteta izvođenja svih vrsta radova, kontroli građevinske dokumentacije i praćenju radova u skladu sa njom, kao i rešavanju novonastalih situacija tokom izvođenja radova.

PUT INŽENJERING

Put inženjering d.o.o punih 25 godina radi kao specijalizovano preduzeće za izgradnju infrastrukture u niskogradnji i visokogradnji, kao i proizvodnjom kamenog agregata i betona. Preduzeće se bavi i transportom, uslugama građevinske mehanizacije i specijalne opreme.

Za spravljanje betona koristimo drobljeni krečnjački agregat sa našeg kamenoloma, deklarisanih frakcija, kontrolisane vlažnosti. Kompletan proces proizvodnje i kontrole kvaliteta vršimo prema važećim standardima.

Kao generalni izvođač radova, vršimo koordinaciju svih učesnika na projektu, planiranje, praćenje i nabavku materijala, kontrolu kvaliteta izvedenih radova, poštujući zadate vremenske rokove i finansijski okvir investitora.

Obradu armature vršimo brzo, stručno i kvalitetno, sa kompjuterskom preciznošću i dimenzijama po projektu.

Osnovi princip našeg poslovanja zasniva se na individualnom pristupu svakom klijentu i pronalaženje najoptimalnijeg rešenja za njegove transportne i logističke potrebe.

Koristeći inovativne tehnike i kvalitetan građevinski materijal iz sopstvenih resursa, spremni smo da odgovorimo na mnoge zahteve naših klijenata iz oblasti niskogradnje.

Naša kompanija u oblasti visokogradnje primenjuje sistem prefabrikovnih betonskih elemenata koji u odnosu na klasičnu gradnju ima brojne prednosti.

Usluge građevinskom mehanizacijom vršimo tehnički ispravnim mašinama, sa potrebnim sertifikatima kako za rukovaoce građevinskim mašinama tako i za same mašine.

Osnovna prednost prefabrikovane konstrukcije jeste brzina kojom konstrukcija može biti projektovana, proizvedena, transportovana i namontirana.

Prednapregnute šuplje ploče su konstruktivni elementi visokog kvaliteta, proizvedeni u fabrički kontrolisanim uslovima.

Raspolažemo opremom i mašinama za sve zemljane radove, kipere i dampere za rad u teškim terenskim uslovima, automiksere i pumpe za beton, autodizalice, podizne platforme.

Izvodimo hidrograđevinske radove u izgradnji kanalizacionih mreža za odvođenje atmosferskih, otpadnih i upotrebljenih voda, izvođenjem hidrograđevinskih radova u okviru regulacije rečnih tokova, kao i izvođenjem hidrotehničkih objekata.

Izrađujemo betonske "New Jersey profile" koji se u svetu koriste za preusmeravanje saobraćaja i zaštitu pešaka u toku izgradnje puta, kao i Betonblock sistem betonskih blokova.

Sakupljanje i privremeno skladištenje otpada vršimo našim specijalizovanim vozilima i deponujemo na našu lokaciju sa odgovarajućom dozvolom. Kapacitet mašine je 250 t/h građevinskog neopasnog otpada.

Površinski kop udaljen je 35 km od Niša. Savremene drobilice, postrojenje za separaciju i sejalica efikasno usitnjavaju i razdvajaju kamene agregate po veličinama. Tehnički kapacitet trenutne primarne drobilice je 300 t/h.

Uslugu transporta vršimo automikserima, kapaciteta bubnja od 7 m³ do 10 m³ betonske mase. Za ugradnju betona posedujemo auto-pumpu za beton, radnog učinka 150 m³/h, sa dužinom strele od 36 m.

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Panelna oplata za ploče Dokadek 30 – Evolucija u sistemima oplate za ploče
Dokadek 30 je ručna oplata lake čelične konstrukcije, bez nosača sa plastificiranim ramovima, koji su prekriveni kompozitnim drveno-plastičnim panelom površine do 3 m^2.

Izuzetno brzo, bezbedno i lako postavljanje oplata
- Mali broj delova sistema i pregledna logistika uz samo dve veličine panela (2,44 x 1,22 m i 2,44 x 0,81 m)
- Dovoljan 2-člani tim za jednostavnu i brzu montažu elemenata sa tla bez merdevina i bez krana
- Sistemski određen položaj i broj podupirača i panela, unapred definisan redosled postupaka
- Prilagođavanje svim osnovama zahvaljujući optimalnom uklapanju sa Dokaflex-om
- Specijalni dizajn sprečava odizanje panela pod uticajem vetra
- Horizontalno premeštanje do 12 m^2 Dokadek 30 pomoću DekDrive

Više informacija o sistemu naći ćete na našem sajtu **www.doka.rs**

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