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Performance of Concrete Beams Reinforced with Various Ratios of Hybrid GFRP/Steel Bars

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Abstract

This paper aims to study the flexural behavior of concrete beams reinforced with hybrid combinations of GFRP/steel bars. To this purpose an experimental program was carried out on four concrete beams reinforced with Glass Fiber Reinforced Polymer (GFRP) and twelve hybrid GFRP/steel Reinforced Concrete (RC) beams. Flexural behavior of the tested beams such as stages of response, failure modes, crack patterns, stiffness, toughness and ductility were analyzed. The experimental results showed that depending on GFRP/steel reinforcement configurations, the behavior of hybrid GFRP/steel RC beams undergoes three or four stages, namely: pre-cracking stage; after concrete cracking and before steel yielding; post-yield stage of the steel bar until peak load and failure stage. Totally six failure modes of hybrid RC beams are reported depending on reinforcement rations and configuration. The effect of reinforcement configuration and ratio of GFRP to steel ($\rho_g$) on the crack patterns, stiffness, ductility and toughness of hybrid RC beams are significant. Based on the non-linear deformation model, an analytical model has been developed and validated to determine the steel yielding moment and ultimate moment of hybrid GFRP/steel RC beams. It could be seen that the experimental values were in good agreement with the predicted values.

Keywords: Concrete Beam; Hybrid Reinforcement; Flexural Strength; Failure Mode; Fiber Reinforced Polymers.

1. Introduction

Damage, reduced service life and failure of concrete structures reinforced with steel bars are inevitably the most common consequences of steel reinforcement corrosion. Chloride ions, present in marine environment and seawater, are considered the main external agent to damage RC structures. The use of nonmetallic reinforcement, i.e. Fiber Reinforced Polymer (FRP), as an alternative reinforcement in concrete structures has emerged as a new solution owing to their non-corrosive and non-magnetic properties. However, due to the low modulus of elasticity of FRP, especially GFRP, there is a significant decrease in the bending stiffness of concrete members reinforced with FRP bars. Furthermore, the fact that the stress-strain relationship of GFRP bars is linear up to failure leads to a brittle failure mode of GFRP RC beams without warning. In order to overcome the drawbacks of individual use of the GFRP reinforcement, the hybrid combination of GFRP and steel reinforcement was introduced in concrete structures. These members could be divided into two groups: members created from existing concrete structures strengthened with GFRP bars; members formed by using simultaneously steel and GFRP bars in new-built concrete structures. For later case, GFRP rebars are placed closer to the concrete surface of the tensile zone with a small cover thickness whereas steel bars are placed with relatively larger concrete cover for a better protection against corrosion. Otherwise, the
brittle behavior and low modulus of elasticity of GFRP reinforcement can be compensated by the presence of steel bars that can improve the ductility of structures.

Regarding the flexural behavior of concrete beams internally reinforced with hybrid reinforcement, Leung and Balendran [1] tested totally five hybrid over-reinforced concrete beams with two different concrete compressive strengths of 30 MPa and 50 MPa. The results indicated that the over-reinforced beams embedded with hybrid reinforcements introduced typical concrete crushing failure mode and no significant change in deflection at failure time. In the literature [2], Qu Wenjun et al. took out experimentally and theoretically investigation on the load-deflection behavior of concrete beams reinforced with hybrid GFRP/steel bars. Their theoretical model was based on the moment-curvature relationship of cross sections to predict the load-deflection relationship of the beams. They suggested that to estimate accurately the flexural performance of hybrid concrete beams, the tension stiffening effect should be included in the theoretical model at various load levels. Lau and Pam [3] investigated the ultimate strength and the ductility of hybrid GFRP/steel RC beams. The experimental results reported that the flexural ductility of pure GFRP RC members can be enhanced by two methods: by increasing the degree of over-reinforcement and by adding conventional steel rebars. To study the contribution of steel bars to behavior of hybrid GFRP/steel concrete beams, Mustafa and Hilal [4] used the nonlinear finite element program (ANSYS software). The results showed that the effect of steel bars on the behavior of hybrid concrete beams is considerably positive. The presence of steel bars improved beam ductility as well as the ultimate capacity after cracking. Similar to the researches mentioned above, the failure mode of almost hybrid concrete beams is first yielding in steel bars then crushing in concrete of compression zone. This is a typical failure mechanism found in previous many studies [1, 5, 6]. In fact, there may still be several modes of failure in hybrid GFRP/steel concrete beams that have not been reported and this study fills the gap by focusing on those failure modes.

To assess the ductility of reinforced concrete structures, it is common to use displacement ductility (μd) or curvature ductility (μΦ). However, because the mechanical properties of steel and GFRP are very different, evaluating the ductility of GFRP/steel hybrid reinforced concrete members by using displacement ductility or curvature ductility may lead to inconsistent outcomes. An energy-based ductility assessment approach can solve this problem. For instance, Bui et al. [7] evaluated the ductility of the FRP/steel RC beams considering the effects of the FRP on the steel reinforcement ratio and effects of the location of the FRP reinforcement on the mechanical performance of the beams. The results of this research showed that the ductility defined by absorption energy was different from that using the ductility factor defined as the conventional steel RC beams since the post-yield behavior of hybrid FRP/steel beams was almost decided by FRP reinforcement. The experimental results of Maranan et al. [8] also depicted that there was a contradiction of ductility between using the displacement ductility and energy-based ductility, the increase in displacement ductility could result in the decrease in energy absorption. Therefore, more research on ductility of concrete members reinforced with FRP/steel reinforcement is still necessary to cover this aspect. In addition, to the best of authors’ knowledge the ductility classification of hybrid GFRP/steel concrete beams has not been mentioned.

At present, the majority of researches on flexural tests of hybrid GFRP/steel RC beams mainly focuses on evaluating bearing capacity load and deflection under service loading. Several studies on establishing the model for predicting moment capacity, deflection, curvature of hybrid FRP/steel RC beams and crack widths were presented [6, 9-11]. These researches revealed that the GFRP to steel ratio affects importantly the flexural performance and crack development of hybrid reinforced concrete beams. Recently, Sun et al. [12] carried out an experimental work on concrete beams reinforced with BFRP/steel in different arrangement of the longitudinal bars, i.e. bundled and distributed reinforcements. The results indicated that the secondary stiffness of beams still increase steadily. However, the effect of FRP to steel ratio on secondary stiffness of the tested specimens was not considered. Furthermore, the other key aspects related to bending behavior such as ductility based on energy, toughness have received little attention in the literature.

In this paper, to identify the failure modes of hybrid GFRP/steel RC concrete beams, twelve hybrid GFRP/steel RC beams and four GFRP RC beams with different reinforcement configurations were tested first. Then, the relationship between midspan deflection and load, stages of flexural behavior, failure modes, crack patterns at failure, stiffness, ductility index, toughness, load-carrying capacity and steel yielding load were analyzed. Finally, the predicted model of flexural behavior adopted from deformation models of materials was introduced and compared with the experimental results.

The rest of the article is structured as follows: Section 2 describes the experimental program of the hybrid GFRP/steel and GFRP RC beams; Section 3 presents test results and discussion in terms of global behavior, crack patterns, flexural stiffness, ductility index, toughness and moment carrying capacity in detail; Section 4 illustrates an analytical model to estimate the steel yielding moment and moment carrying capacity of hybrid beams; and the conclusions are given in the final section. The research flow chart is shown in Figure 1.
2. Experimental Study

2.1. Specimen Details

The testing beams were all designed as simply supported beams with a rectangular cross-section (150×250 mm). The total length \(l\) of the beam was 2700 mm. The testing span \(l_0\) was 2400 mm, of which the pure bending length was 400 mm (Figure 2).
The concrete beams reinforced with GFRP and hybrid GFRP/steel reinforcement were designed with reference to ACI 440.1R-15 [13]. In hybrid GFRP/Steel RC beams, the GFRP bars are located closer to the surface with the cover thickness \((C_g)\) of 20 mm while the steel rebars are located deeper with the cover thickness \((C_s)\) of 50 mm (Figure 3). Two steel bars of 6-mm diameter were used as reinforcement at the compression zone with concrete cover of 20 mm. The stirrups were made of steel plain round bars with a diameter of 6 mm, which had 100 mm spacing in shear span to avoid shear failure and a 200 mm spacing in midspan. The deformed steel bars with diameters of 10 mm, 12 mm, 14 mm and the GFRP bars with transverse spiral grooves with diameters of 10 mm, 12 mm and 14 mm are used as tensile reinforcement.

The beams were designed so that all possible failure modes may occur except the case of steel over reinforcement. The testing beams varied in the GFRP and steel reinforcement ratios and were divided into four groups. In each group, the GFRP reinforcement ratio \(\mu_g\) was fixed (Group #1 – 2G10; Group #2 – 2G12; Group #3 – 2G14 and Group #4 – 3G14) and steel reinforcement ratios \(\mu_s\) increased from about 0% to 1.13%. According to preliminary calculation results following ACI 440.1R-15 [13], the balanced reinforcement ratio for GFRP RC beam was 0.4%, hence the
GFRP reinforcement ratios used for testing beams varied from 0.36% to 1.16%. The actual dimensions, the concrete covers and reinforcement ratios of each beam will be measured and determined at the time of casting and experiment. Details of testing beams are given in Table 1. During the analysis of the experimental results, to evaluate the effect of the GFRP reinforcement on the characteristics of beams, the groups of hybrid RC beams with fixed steel reinforcement and varying GFRP reinforcement will be created: group of beams 2S10 (2G10-2S10; 2G12-2S10; 2G14-2S10 and 3G14-2S10); group of beams 2S12 (2G10-2S12; 2G12-2S12; 2G14-2S12 and 3G14-2S12); group of beams 2S14 (2G10-2S14; 2G12-2S14; 2G14-2S14 and 3G14-2S14) and group of beams S0 (2G10-S0; 2G12-S0; 2G14-S0 and 3G14-S0).

Table 1. Details of testing beams

| Group of beams | Beam ID | b, mm | h, mm | C_s mm | C_g mm | h_{0s} mm | h_{0g} mm | A_s cm^2 | A_g cm^2 | R_m, MPa | μ_s% | μ_g% | μ_t% |
|---------------|--------|------|------|-------|-------|---------|---------|--------|--------|--------|-----|-----|-----|
| #1 (2G10)     | 2G10-S0 | 151  | 253  | 21    | -     | 227     | -       | 1.23   | -      | 39.0   | -   | 0.36 | 0.36 |
|               | 2G10-2S10 | 152  | 254  | 25    | 49    | 224     | 200     | 1.23   | 1.57   | 41.6   | 0.52 | 0.36 | 0.88 |
|               | 2G10-2S12 | 155  | 252  | 20    | 53    | 227     | 193     | 1.23   | 2.26   | 43.1   | 0.76 | 0.35 | 1.11 |
|               | 2G10-2S14 | 150  | 253  | 28    | 54    | 220     | 192     | 1.23   | 3.08   | 39.2   | 1.07 | 0.37 | 1.44 |
| #2 (2G12)     | 2G12-S0  | 151  | 249  | 16    | -     | 227     | -       | 1.87   | -      | 44.2   | -   | 0.55 | 0.55 |
|               | 2G12-2S10 | 150  | 250  | 25    | 50    | 219     | 195     | 1.87   | 1.57   | 41.0   | 0.54 | 0.57 | 1.11 |
|               | 2G12-2S12 | 155  | 250  | 20    | 45    | 224     | 199     | 1.87   | 2.26   | 37.4   | 0.73 | 0.54 | 1.27 |
|               | 2G12-2S14 | 151  | 248  | 23    | 55    | 219     | 186     | 1.87   | 3.08   | 38.6   | 1.10 | 0.57 | 1.67 |
| #3 (2G14)     | 2G14-S0  | 150  | 256  | 31    | -     | 218     | -       | 2.65   | -      | 44.0   | -   | 0.81 | 0.81 |
|               | 2G14-2S10 | 151  | 251  | 24    | 47    | 220     | 199     | 2.65   | 1.57   | 41.0   | 0.52 | 0.8  | 1.32 |
|               | 2G14-2S12 | 148  | 250  | 16    | 48    | 227     | 196     | 2.65   | 2.26   | 44.3   | 0.78 | 0.79 | 1.57 |
|               | 2G14-2S14 | 153  | 255  | 26    | 55    | 221     | 192     | 2.65   | 3.08   | 43.2   | 1.06 | 0.79 | 1.85 |
| #4 (3G14)     | 3G14-S0  | 148  | 255  | 19    | -     | 229     | -       | 3.97   | -      | 45.5   | -   | 1.17 | 1.17 |
|               | 3G14-2S10 | 152  | 254  | 28    | 47    | 219     | 202     | 3.97   | 1.57   | 40.9   | 0.51 | 1.19 | 1.70 |
|               | 3G14-2S12 | 152  | 254  | 23    | 48    | 224     | 200     | 3.97   | 2.26   | 41.0   | 0.74 | 1.16 | 1.90 |
|               | 3G14-2S14 | 155  | 255  | 16    | 51    | 232     | 197     | 3.97   | 3.08   | 42.9   | 1.01 | 1.11 | 2.12 |

Note: 1) The capital letters G and S denote the steel bar and GFRP bar respectively; b and h – the average width and height of the cross section at midspan respectively; 2) The notation of 2G10-2S12 represents the beam’s reinforcement with 2 GFRP bars in diameter of 10 mm and 2 steel bars in diameter of 12 mm; 3) C_s and C_g are the distances from bottommost concrete fiber to the centroid of GFRP and steel bars (concrete covers) respectively; h_{0s} and h_{0g} are the distances from outermost compressive concrete fiber to the centroid of GFRP and steel bars respectively; A_s and A_g are the areas of the cross section of GFRP and steel bars respectively; μ_s=A_s/(b×h_{0s}); μ_g=A_g/(b×h_{0g}) and μ_t=μ_s+μ_g are the steel, GFRP and total reinforcement ratio respectively.

The average cubic compressive strength R_m of concrete of each beam is evaluated by test on six cubic specimens (150x150x150 mm) after 28-days of curing (Table 1). GFRP bars used for the experiment were manufactured by Vietnam Fiber Reinforced Polymer Products, JSC. The average tensile strength and tensile modulus of elasticity of GFRP bars are 970 MPa and 44300 MPa respectively [14]. The deformed steel bars have average yield strength σ_y of 412 MPa, ultimate tensile strength σ_u of 577 MPa and modulus of elasticity E_s=200 GPa. Modulus of elasticity E_p, prismatic strength R_p and tensile strength R_{ts} of concrete are determined empirically through the cubic strength: R_m=0.8R_{ps}; R_{ps}=5R_m/(45+R_m), MPa and E_p=55000R_m/(27+R_m), MPa [15]. The experimental tensile stress-strain relationships of steel and GFRP bar are illustrated in Figure 4.

2.2. Test Setup and Instrumentation

All beams were tested up to failure under a monotonic load in four-point bending test as shown in Figure 2. One LVDT that measures the midspan deflection was placed in the midspan, and two dial indicators I1 and I2 were placed at both ends of the beam to eliminate holder deformation. Load on beams from hydraulic jack is recorded by a 300 kN loadcell. Data from LVDT and loadcell are collected by Wireless Wi-Fi Data Logger STS-WIFI. Data from dual indicators are collected with naked eyes.

Before loading on beam, a 2 kN preloading step was performed to check and eliminate the errors of the instruments at the initial stage of loading. The load applied on beam was gradually increased by steps. The rate of the actuator was set to 3 kN/min. during load control and 2 mm/min. during displacement control. At each level of loading, the load was held constant for approximately 10-15 seconds for recording data from dual indicators and drawing crack patterns. The propagation of cracks was marked by a color marker directly on each beam.

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# 3. Test results and Discussion

## 3.1. Analysis of Behavior Stages and Failure Process

The load-deflection curves of all tested hybrid GFRP/steel RC beams (Figure 5) show that the behavior of hybrid GFRP/steel RC beams from start of loading until complete failure could be divided into three stages or four stages depending on reinforcement ratios and configurations: first stage - pre-cracking stage (elastic stage); second stage - after concrete cracking and before steel yielding; third stage - post-yield stage of the steel bar until peak load and fourth stage – failure stage (after peak load).

### Notes:
- SY – steel yielding; RG - rupture of GFRP; RS - rupture of steel; CC - concrete crushing

**Figure 5. Load-deflection curves of tested beams**

In the first stage, the materials behave elastically, so load-deflection relationships of tested beams are linear. At the end of this stage, the deflections of the tested beam vary from 1.53 mm to 2.63 mm, which are 1.91-5.09% of deflections at peak point (Table 2). After concrete cracked, the stiffness of the beams decreases, deflections rapidly increase. This is shown by the decrease of the slopes on load-deflection curves. In this second stage, the load-deflection relationship is approximately linear, so the stiffness of tested beams in this stage is nearly stable. At the steel yielding point, the deflections of tested beams vary from 8.9 mm to 12.7 mm (Table 2), which are about 13.5-31.15% of deflections at peak point. In the third stage, after the steel bar yielded, the stiffness of the hybrid RC beam is further reduced, and the midspan deflection rises rapidly. It can be seen on Figure 5 that at this stage the load-deflection relationships of tested beams are almost linear, so the stiffness of hybrid RC beams in this stage also remains stable. At the end of the third stage, if one of the strains in the GFRP bar or in the outmost compressive concrete fiber reaches its limit value, the beam will be collapsed. It is noted that the rupture strain of steel bar is much higher than that of GFRP, so the rupture of GFRP occurs before the rupture of steel bar. In hybrid RC beams with low GFRP and steel reinforcement ratios (with lack of reinforcements), at the end of the third stage, strain of GFRP bar
reaches the limit value. Once the GFRP has failed, the tensile stress of steel bars would immediately surpass the ultimate stress value then the beams would breakdown without any forewarning (beam 2G10-2S10). In this case, the behavior of hybrid RC beam undergoes only the first three stages. For the beams with high GFRP and steel reinforcement ratios, after the third stage (at the peak load) the beam will be damaged by rupture of GFRP or crushing of concrete in the compression zone (beams 2G10-2S14 and 2G12-2S14). After that the load will decrease gradually and the deflection will increase rapidly. In this case, after peak load, the beams are still able to remain a part of applying load, and the behavior of the hybrid RC beams is divided into four stages. The fourth stage is characterized by the descending branch of load-deflection curves.

### Table 2. Testing results

| Group of beams | Beam ID | Failure mode | Cracking point | Yielding point | Peak point | Ultimate point |
|---------------|---------|--------------|----------------|----------------|-----------|----------------|
|               |         |              | \( P_c, kN \) | \( f_c, mm \) | \( P_y, kN \) | \( M_y, kNm \) | \( f_y, mm \) | \( M_u, kNm \) | \( f_u, mm \) |
| #1 (2G10)     | 2G10-S0 | RG           | 10.6           | 1.93           | -          | -               | 46.0           | 23.0           | 56.53           | 56.53           |
|               | 2G10-2S10 | SY-RG and RS | 12.4           | 1.38           | 31.4       | 15.7           | 8.92           | 68.7           | 34.4           | 57.54           | 56.53           |
|               | 2G10-2S12 | SY-RG and CC | 13.9           | 2.39           | 42.2       | 21.1           | 11.21          | 71.0           | 35.5           | 55.49           | 68.00           |
|               | 2G10-2S14 | SY-RG and CC | 12.4           | 1.80           | 52.3       | 26.2           | 11.20          | 75.9           | 38.0           | 42.09           | 54.87           |
| #2 (2G12)     | 2G12-S0 | CC           | 13.1           | 1.85           | -          | -               | 77.1           | 38.6           | 70.95           | 78.39           |
|               | 2G12-2S10 | SY-CC       | 11.6           | 2.16           | 35.2       | 17.6           | 8.90           | 82.5           | 41.3           | 65.98           | 83.19           |
|               | 2G12-2S12 | SY-CC       | 14.4           | 1.72           | 49.7       | 24.9           | 10.53          | 91.2           | 45.6           | 53.90           | 78.15           |
|               | 2G12-2S14 | SY-CC       | 13.0           | 2.08           | 58.4       | 29.2           | 12.70          | 89.9           | 45.0           | 44.85           | 55.92           |
| #3 (2G14)     | 2G14-S0 | CC           | 13.0           | 2.53           | -          | -               | 90.1           | 45.1           | 69.04           | 82.87           |
|               | 2G14-2S10 | SY-CC       | 14.2           | 2.40           | 39.7       | 19.8           | 10.28          | 96.8           | 48.4           | 57.57           | 75.61           |
|               | 2G14-2S12 | SY-CC       | 12.7           | 2.10           | 49.2       | 24.6           | 10.74          | 91.5           | 45.8           | 42.40           | 78.59           |
|               | 2G14-2S14 | SY-CC       | 13.0           | 1.26           | 64.5       | 32.3           | 10.50          | 113.0          | 56.5           | 44.42           | 59.28           |
| #4 (3G14)     | 3G14-S0 | CC           | 13.3           | 1.81           | -          | -               | 104.7          | 52.4           | 58.69           | 59.92           |
|               | 3G14-2S10 | SY-CC       | 14.3           | 2.22           | 46.8       | 23.4           | 10.49          | 98.8           | 49.4           | 43.60           | 62.75           |
|               | 3G14-2S12 | SY-CC       | 14.1           | 1.89           | 56.1       | 28.1           | 10.88          | 106.1          | 53.1           | 42.43           | 56.67           |
|               | 3G14-2S14 | SY-CC       | 13.6           | 1.55           | 69.0       | 34.5           | 11.56          | 113.4          | 56.7           | 37.11           | 43.75           |

Note: \( P_c \) and \( f_c \) are the load and deflection at cracking point respectively; \( P_y \) and \( f_y \) are the load and deflection at steel yielding point respectively; \( P_u \) and \( f_u \) are the load and deflection at ultimate point, which is equal to 85% of the peak load on the descending branch; \( M_y \) and \( M_u \) – the steel yielding moment and moment-carrying capacity of the beams (\( M_y=P_c \times \sigma_0 \) and \( M_u=P_u \times \sigma_0 \)).

Similar analysis of the GFRP RC beams indicates that the beams undergo only two stages (pre-cracking and after concrete cracking – beam 2G10-S0) or three stages (pre-cracking, after concrete cracking and failure – beams 2G12-S0, 2G14-S0 and 3G14-S0).

Depending on reinforcement configuration and the ratio of reinforcements \( \rho_{GR} \), the failure modes of hybrid GFRP/steel RC beams vary. Some of the failure modes of hybrid GFRP/steel RC beams were reported in the previous studies. Based on the above-analyzed experimental results and results published in literature, the following six failure modes of hybrid GFRP/steel can be drawn. Mode 1 - steel yielding, GFRP rupturing and then steel rupturing immediately, concrete non-crushing (beam 2G10-2S10). This is a brittle failure that is similar to the under-reinforced concrete beams. This failure mode is also mentioned in [5, 16, 17]. When the strain of GFRP rebars reaches the ultimate value, the steel rebars have already yielded as their yielding strain is smaller than the ultimate strain of the GFRP. This mode is a brittle failure and is therefore not suggested in practice. Mode 2 - steel yielding, GFRP rupturing and sequentially concrete crushing (beams 2G10-2S12 and 2G10-2S14). In this case, first, steel yields and then GFRP ruptures, which involve the crush of concrete in compression zone. The breakdown of beams occurs without rupture of steel bars. Mode 3 - steel yielding, concrete crushing, FRP non-rupturing. This failure mode occurs in the remaining tested hybrid RC beams and also reported in [5, 6, 9, 17, 18]. Mode 4 - concrete crushing without yielding of steel as reported in [5, 6, 9, 16, 17, 18]. This failure mode may occur if the hybrid member reinforced with too much reinforcement and is not allowed in practical structure for its brittle failure. Mode 5 - steel yielding, rupture of GFRP and concrete crushing simultaneously (balanced reinforced) as recorded in [11]. Mode 6 - steel yielding and concrete crushing simultaneously (balanced reinforced) [11]. The failure modes of tested GFRP RC beams are rupture of GFRP (beam 2G10-S0) and crush of concrete (beams 2G12-S0; 2G14-S0 and 3G14-S0). Details of failure modes of tested beams obtained from experiment are shown in Table 2 and Figure 6.
3.2. Analysis of Crack Patterns at Failure

Crack patterns of the tested beams were observed and redrawn after each step of loading. After concrete cracking, the number of cracks in hybrid RC beams developed. After yielding of steel the existing cracks propagate until the beam is broken and almost no new cracks appear. Crack patterns after failure of tested beams are illustrated in Figure 6. The overall distribution of the cracks is relatively uniform and symmetric over a length of 1700 mm to 1980 mm in the middle of the beams. The numbers of minor and major cracks and the average minor and major crack spacing of tested beams at failure are shown on Figure 7. Testing results show that when increasing the total reinforcement ratio, i.e. increasing the number of longitudinal bars, the number of major and minor cracks expands, the crack distribution length enhances, the average minor and major crack spacing reduce. This phenomenon is explained by higher bond strength when increasing the number of longitudinal bars. These experimental results are consistent with the results of previous studies. Aiello and Ombres [19] tested hybrid GFRP/steel RC beams with span 2700 mm, pure bending zone 100 mm and reported that at failure the major crack spacing was about from 60 mm to 100 mm and the number of cracks was from 25 to 30. The study results of Ge W. et al. [18] indicated that the average crack spacing of hybrid BFRP/steel RC beams at failure was from 93 mm to 108 mm.

![Figure 6. Crack patterns and failure modes of tested beams](image)

![Figure 7. Number of cracks and average crack spacing at failure](image)

3.3. Analysis of Secondary Stiffness and Ductility Index

Figure 8 shows the schematic diagrams for determining the stiffness and ductility of tested hybrid RC beams based on...
on the experimental load-deflection curve. In this study, initial equivalent stiffness $K_1$ (before steel yielding) and secondary stiffness $K_{II}$ (after steel yielding) of hybrid GFRP/steel RC beams were calculated and analyzed. These stiffnesses can be defined as follows [12]:

$$K_1 = P_y / f_y$$

$$K_{II} = \frac{P_{cr} - P_y}{P_{cr} - f_y}$$

Where: $P_y$, $f_y$, and $P_{cr}$ are listed in Table 2.

Secondary stiffness ratio of the tested hybrid RC beams is calculated by following equation:

$$r_b = K_{II} / K_1$$

Flexural stiffness of RC beams depends mainly on their cross-sectional area and modulus of elasticity of materials. The calculated results in Table 3 and Figure 9 show that in each group of hybrid RC beams with fixing GFRP reinforcement, the initial stiffness of tested beams increases proportionally to the increase in the ratio of steel reinforcement. Similarly, when steel reinforcement is fixed, the initial stiffness of hybrid RC beams also ascends linearly with the GFRP reinforcement ratio. For groups of hybrid RC beams with fixing GFRP reinforcement (Figure 9a and Table 3), when increasing steel reinforcement ratios from 0.51% to about 1.1% the initial equivalent stiffness of groups of beams 2G10, 2G12, 2G14 and 3G14 increases 1.33, 1.22, 1.59 and 1.34 times respectively. In groups of beams with fixing steel reinforcement 2S10, 2S12 and 2S14 (Figure 9b and Table 3), when increasing GFRP reinforcement ratios from 0.35% to 1.19%, the initial stiffness increases 1.27, 1.37 and 1.28 times respectively. These results indicate that in hybrid RC beams influence of steel reinforcement on the initial equivalent stiffness is greater than the effect of GFRP due to the important difference in elastic modulus.

Development of cracks and yielding of steel lead to significant loss of secondary stiffness. The results in Table 3 show that after yielding of steel the secondary stiffness of hybrid RC beams in each group of tested hybrid beams with fixing GFRP reinforcement is almost the same despite steel reinforcement varies, and the average values of stiffness of these groups rise along with the increase of GFRP reinforcement. This outcome indicates that the influence of steel reinforcement after yielding on the stiffness of hybrid RC beams is negligible. Specifically, after steel yielding the stiffness of hybrid RC beams reduces by 65-84% in comparison with the initial stiffness. Secondary stiffness ratio $r_b$ shows the reducing ratio of stiffness after steel yielding to the initial stiffness. For the tested hybrid RC beams, the stiffness ratios vary from 0.16 to 0.35 depending on reinforcement ratio and the secondary stiffness ratio is inversely proportional to the steel reinforcement ratio in each group. As mentioned above, the steel reinforcement significantly influences the stiffness of hybrid RC beams, thus the secondary stiffness ratio increases with the increase of the ratio of reinforcements $\rho_s$ (Figure 10).

![Figure 8. Diagrams of stiffness and ductility indexes](image)

| Group of beams | Beam ID | Reinforcement ratio $\mu_s$ | Stiffness $E_{el}$ kN/mm, $E_{inel}$ kN/mm | Ductility $\mu$ | Toughness $U_{T_p}$, MPa |
|----------------|---------|-----------------------------|---------------------------------------------|--------------|------------------------|
| #1 (2G10)      | 2G10-50 | 0.36                        | 303.9, 1284.1                               | Ductile      | 1.00                   |
|                | 2G10-2S10 | 0.52, 0.36                 | 440.0, 2357.0                               | Ductile      | 6.34                   |
|                | 2G10-2S12 | 0.76, 0.35                 | 472.9, 2459.1                               | Ductile      | 6.07                   |
|                | 2G10-2S14 | 1.07, 0.76                 | 331.0, 2027.0                               | Ductile      | 4.90                   |
| #2 (2G12)      | 2G12-50 | 0.55                        | 734.2, 3220.8                               | Ductile      | 1.10                   |
|                | 2G12-2S10 | 0.54, 0.57                 | 706.7, 3262.3                               | Ductile      | 9.35                   |
|                | 2G12-2S12 | 0.73, 0.60                 | 632.7, 2935.3                               | Ductile      | 7.42                   |
|                | 2G12-2S14 | 1.10, 0.75                 | 459.8, 2471.2                               | Ductile      | 4.40                   |
The ductility of the structure mentions as the inelastic deformation capacity prior to collapse without significant loss of strength. The greater the ductility, the greater the ultimate deformation capacity, and the structure will be relatively safer under the same load. It is well-known that the ductility of reinforced concrete beams is directly related to the amount of tension reinforcement. A higher tensile reinforcement ratio results in less ductile behavior. The ductility can be expressed in terms of displacement or energy absorption. The displacement ductility can be obtained from the ratio of ultimate displacement to the yield displacement. The energy ductility can be defined as the ratio relating any two of the inelastic, elastic energies to total energy (Figure 8b). The conventional ductility or displacement ductility cannot be applied to hybrid RC beams, because it does not include the effect of the secondary stiffness, which leads to increase the ductility of hybrid RC beams [7, 12]. Therefore, in this paper the energy ductility index $\mu$ of tested beams is calculated and analyzed. In addition, as a basis for comparison, the displacement ductility is also calculated (Table 3).

\[ S = \frac{P_1S_1 + (P_2 - P_1)S_2}{P_2} \]  

Where: $P_1$ and $P_2$ – loads as shown in Figure 8b, $S_1$ and $S_2$ – corresponding slopes.

Total energy:

\[ E_{\text{tot}} = E_{\text{inel}} + E_{\text{el}} \]  

The energy ductility index can be calculated as follows [21] and the results are shown in Table 3:

\[ \mu = \frac{1}{2} \left( \frac{E_{\text{tot}}}{E_{\text{el}}} + 1 \right) \]  

The calculated results of energy ductility indexes in Table 3 and on Figure 11a indicate that in each group of beams with fixing the GFRP reinforcement, the ductility indexes of hybrid GFRP/steel RC beams increase with boost of the steel reinforcement ratio and these relationships follow a linear trend. The GFRP reinforcement ratio versus ductility index relationships of these groups is illustrated in Figure 11b. It can be seen that an increase in GFRP ratio reduces the ductility index of tested beam and these relationships also have linear tendency.

The ductility of RC beams relates to the shape of the load-displacement relationship of the sections, which is mainly decided by the ratio of steel reinforcement to GFRP reinforcement and total reinforcement. From Table 3 and Figure 11c it can be seen that the ductility indexes of hybrid RC beams are inversely related to the ratio of...
reinforcement $\rho_s$. Simultaneously, with the same ratio $\rho_s$, the ductility index is inversely proportional to the total reinforcement ratio. For example, beams 2G10-2S10, 2G12-2S12 and 2G14-2S14 have nearly the same values of ratio $\rho_s$ when total reinforcement ratios vary from 0.87% to 1.85% (ratio 1:1.46:2.12), the ductility indexes reduce from 3.68 to 2.38 (ratio 1.5:0.9:0.65). Similarly, the above finding can be verified with pairs of beams 2G12-2S10 and 2G14-2S12 or 2G14-2S10 and 3G14-2S12. When increasing the steel reinforcement ratios from 0% to about 1.1%, the ductility indexes of groups of hybrid RC beams reinforced with 2G10, 2G12, 2G14 and 3G14 increase by 1.31, 1.16, 1.2 and 1.2 times. For groups of beams with fixing steel reinforcement (Group S0 - without steel reinforcement, group 2S10, group 2S12 and group 2S14), when GFRP reinforcement ratios increase from 0.36% to 1.19%, the ductility indexes decrease respectively 1.92, 2.16, 1.91 and 2.09 times.

Figure 11. Energy ductility indexes of tested beams

With the purpose to classify the tested beams according to their ductility, the energy ratio is defined as the ratio of the inelastic energy to total energy ($E_{\text{inel}}/E_{\text{tot}}$). In the light of this study, if the displacement ductility requirement of energy ductility index is greater than 3 or corresponding energy ratio $E_{\text{inel}}/E_{\text{tot}}$ is more than 80%, the authors suggest that the hybrid beams will exhibit a ductile failure. On the other hand, when the energy ductility index ranges between 2 and 3, the beams will be considered to be less ductile behavior. In another case, the beam experiences a brittle failure if the energy ratio is below 65% or energy ductility index is less than 2. Ductility classification of the tested beams according to their ductility is listed in Table 3. Table 3 depicts that all beams of group #1 and #2 except for the beams 2G10-S0 and 2G12-S0, present not only the energy ductility index greater than 3 but also present $E_{\text{inel}}/E_{\text{tot}}$ ratio greater than 80%. In this case, two beams of 2G10-S0 and 2G12-S0 indicate that the inelastic energies consumed prior to failure, $E_{\text{inel}}$ accounts for more than 80% of total energies but the displacement ductility $\mu_s$ is even equal to 1.0 and 1.10, respectively. This finding is in agreement with Maranan et al.’s data [8]. It can be seen that for groups of beams #3 and #4, whose amount of GFRP ratios are nearly twice and three times higher than the balanced GFRP ratio, the $E_{\text{inel}}/E_{\text{tot}}$ ratios have dropped considerably by 30% and 40% respectively in comparison with groups of beams #1 or #2. It is obvious that the high GFRP ratios in hybrid beams reduce considerably the energy dissipation capacity after yielding of hybrid beams.

3.4. Analysis of Toughness

The toughness $U_T$ is the ability of a material to absorb energy and plastically deform without fracturing [22]. This parameter can be calculated by dividing total area below the stress–strain curve or the force–deformation curve by the volume of the tested beam. In this work, the force–deformation curves are used for determining the toughness at peak load ($U_{T,p}$). The calculated values of toughness of tested beams are listed in Table 3 and compared on Figure 12 for each group of beams.

Figure 12. Toughness of tested beams

It can be seen in Figure 12a, in group with fixing the steel reinforcement, the toughnesses of GFRP RC and hybrid RC beams at peak point increase when increasing the GFRP reinforcement ratio to 0.8%. Further increasing GFRP...
reinforcement ratio will lead to a reduction in this toughness. In groups of beams with fixing GFRP reinforcement (Figure 12b), the changing tendency of toughness at peak point is various. In the group of beams #1 (2G10), the toughness at peak point and ultimate point enhances when increasing steel reinforcement ratio to 0.6%. After that the toughness at peak point tends to decrease when the steel reinforcement ratio increases from 0.6 to 1.1% (Figure 12b). Meanwhile, for groups of beams #2 (2G12), #3 (2G14) and #4 (3G14), the toughness at peak point tends to decrease with the increase of steel reinforcement ratio. Moreover, the toughness of groups #4 at ultimate point is around 35% lower than that of groups #3 since the GFRP ratio is approximately three times higher than the balanced GFRP ratio. The reason is that increasing steel reinforcement in groups of beams #2 (2G12), #3 (2G14) and #4 (3G14) reduces significantly deflections at peak point, meanwhile, the peak loads of the beams in each of these groups vary insignificantly due to failure by crush of concrete in the compression zone.

3.5. Analysis of Steel Yielding Moment and Moment-carrying Capacity

The steel yielding load listed in Table 2 is determined by the load at the point of sudden change in deflection on load-displacement curve (Figure 5). To clarify the effect of GFRP reinforcement on steel yielding moment ($M_y$) in each group of beams, the steel yielding moment evolution as a function of the GFRP reinforcement ratio was build and shown in Figure 13. Before yielding of steel, the GFRP and steel reinforcements receive the tensile force. So, increasing GFRP reinforcement ratio boosts steel yielding load and the GFRP reinforcement ratio versus steel yielding moment relationships are linear (Figure 13). At the same time, GFRP reinforcement ratio-steel yielding moment relationships of all groups of beams with fixing steel reinforcement ratio have the same tendency. Specifically, when increasing GFRP reinforcement ratio from 0.35% to 1.19%, the steel yielding moment of groups 2S10, 2S12 and 2S14 enhances 1.37, 1.33 and 1.32 times, respectively.

Observing the load-deflections in Figure 5, after steel yielding, due to the large ultimate strain of GFRP the hybrid RC beams continues to receive the load. The load gain carried by the hybrid RC beams between the steel yielding point and the peak point depends mainly on the steel reinforcement ratio, GFRP reinforcement ratio and concrete strength. According to the experimental results in Table 3, with the same GFRP reinforcement ratio, the load gain between the steel yielding point and peak point of hybrid RC beams reduces when increasing steel reinforcement ratio. On the contrary when fixing steel reinforcement load gain increases proportionally to GFRP reinforcement ratio.

The experimental load-carrying capacity of the tested beams listed in Table 2 is determined at the peak point of load-displacement curve (Figure 5). The moment-carrying capacity ($M_u$) versus reinforcement ratio relationships for all tested beams is illustrated in Figure 14. In group with fixing steel reinforcement (groups S0; 2S10; 2S12 and 2S14), as the GFRP reinforcement ratio increases from 0.35% to 1.2%, the moment bearing capacity of GFRP RC beams (Group S0) rise to 128.1%, the corresponding values for groups of beams 2S10, 2S12 and 2S14 are 43.8%, 49.4% and 49.4% respectively (Figure 14a). Besides, the Figure 14a shows that the load-carrying capacity of groups of beams with fixing steel reinforcement sharply increases in range of GFRP reinforcement from 0.35% to 0.8%. After that the difference of load-carrying capacities of tested beams is not significant when further increasing GFRP reinforcement ratio (Figure 14a). This can be explained by the failure initiated at concrete compression zone. At GFRP reinforcement ratio about 1.2%, the difference of moment-carrying capacities of tested beams is not significant.

![Figure 13. Relationship between GFRP reinforcement ratio and steel yielding load](image1)

![Figure 14. Moment-carrying capacity versus reinforcement ratio relationships](image2)

To evaluate the effect of steel reinforcement on the moment-carrying capacity of hybrid RC beams, the moment-carrying capacity versus steel reinforcement ratio relationships were presented in Figure 14b. It can be seen on Figure 14b that influence of steel reinforcement on load-carrying capacity of hybrid RC beams is dissimilar. When adding steel reinforcement with the ratio from 0% to 1.07% to GFRP under-reinforced concrete beam 2G10, the moment-carrying capacity increases to 65.4%. Meanwhile, adding steel reinforcement from 0% to about 1.07% to the GFRP over-reinforced beams (2G12, 2G14 and 3G14) leads to an increase in moment-carrying capacity of 16.6%, 25.4 and
8.3%, respectively. With the same total reinforcement ratio, the load-carrying capacity of the hybrid RC beams increase when increasing the ratio $\rho_s$ (for example, the beams 2G10-2S12, 2G12-2S10 and 3G14-50 or the beams 2G12-2S14, 3G14-2S10).

Depending on the reinforcement configuration, the ratio of the steel yielding moment to moment capacity of the tested beams ($M_y/M_a$) varies from 0.41 to 0.69. In each group of beams with fixing GFRP reinforcement ratio the ratios of $M_y/M_a$ improve when increasing steel reinforcement ratio. It is noticed from experimental results that with the same total reinforcement ratio, the steel yielding moment is proportional to the ratio $\rho_s$ (pairs of beams: 2G10-2S12 versus 2G12-2S; 2G12-2S10 versus 3G14-2S10; 2G14-2S14 versus 3G14-2S12 etc.).

4. Predicted Model of Moment-carrying Capacity and Steel Yielding Moment

As stated above, several methods of calculating the bearing capacity of beams of hybrid FRP/steel have been proposed by some authors in previous studies. Recently, Kara et al. [9] classified three failure modes and proposed the formulas to predict the flexural capacity for hybrid FRP/steel RC beams based on equilibrium of forces and full compatibility of strains. It was reported that the ratio of predicted value to experimental value of flexural capacity varied in wide range (between 0.77 and 1.25). The reason is probably because the authors employed the inappropriate stress-strain diagrams of materials. Lei Pang et al. [5] considered the strength at initiation of yielding ($M_y$) as the design-aimed state and the difference between the flexural strength at the ultimate state and that at the steel yielding state can be considered as the reserve strength. Then, they proposed the formula to predict the steel yielding moment according to the provisions of ACI 318M-05.

In this paper, the authors introduce another method to determine the steel yielding moment and moment-carrying capacity of hybrid GFRP/steel RC beams based on the deformation models of materials [23]. For GFRP bars, according to the tensile test [14], the stress-strain relationship is linear until failure (Figure 4). Tri-linear simplified stress-strain diagram according to SP 63.13330.2018 [23] is chosen for concrete (Figure 15a). The stress-strain relationship of steel bar is also formulated as a simplified trilinear diagram based on the actual relationship (Figure 15b). All characteristic points in Figure 15 are taken from SP 63.13330.2018 [23].

![Figure 15. Tri-linear simplified stress-strain diagrams of concrete and steel bars](image)

Note: $R_s$ and $R_y$ are the prismatic compressive and tensile strength of concrete respectively; $\sigma_s$ and $\sigma_y$ – yield stress and ultimate stress of steel.

In order to ensure accuracy, in the calculation procedure the compressive reinforcement area ($A_{sc}$) is taken into account. The normal cross-section is divided to $n$ equal segments (layers) in height (Figure 16). Assuming that: “Plane cross-section remains plane before and after bending”; each fiber works under stress conditions in a single axis of stress state.

In calculation, the forces and deformations in a cross-section are determined according to non-linear deformation models using equilibrium expressions of internal and external forces in the section. In addition, distribution of concrete strain and reinforcement strain along the sectional height is assumed according to the linear law (flat cross-section hypothesis). Regarding the algebraic sign convention, “minus” refers to axial compression force, compressive stresses and strains of concrete and reinforcement; “plus” stands for axial tension force, tensile stresses and elongation strains of concrete and reinforcement.

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For bending element in the symmetrical plane of the normal cross section and the X-axis in this plane, the moment equilibrium is as follows:

\[ M = D_{11} \left( \frac{1}{r_x} \right) \]  
\[ (7) \]

Where: \( 1/r_x \) - the curvature of element and \( D_{11} \) - rigidity characteristic of cross section. The values of these parameters are determined as in following equations:

\[ \frac{1}{r_x} = \frac{\varepsilon_{bi}}{Z_{bi}} = \frac{\varepsilon_{sc}}{Z_{sc}} = \frac{\varepsilon_g}{Z_g} \]  
\[ (8) \]

\[ D_{11} = \sum_{i=1,...} A_i Z_i^2 E_i \sigma_i \varepsilon_i + A_i Z_i^2 E_i \sigma_i \varepsilon_i + A_i Z_i^2 E_i \sigma_i \varepsilon_i + A_i Z_i^2 E_i \sigma_i \varepsilon_i \]  
\[ (9) \]

Where: \( \varepsilon_{bi}, \varepsilon_{bi}, \varepsilon_g, Z_{bi}, Z_s, Z_{sc}, Z_g \) shown in Figure 16; \( E_{bi}, E_s \) and \( E_g \) - the modulus’s of elasticity of concrete, steel in tension zone, steel in compression zone and GFRP respectively; \( \nu_i \) - elastic coefficients of concrete in i-layer, steel in tension zone and compression zone, respectively. These values are determined by following formulas:

\[ \nu_i = \frac{\sigma_i}{E_i \varepsilon_i} \]  
For i-layer greater than 0, i.e. tensile strain, use \( \sigma_{bi} \) instead of \( \sigma_i \)  
\[ (10) \]

\[ \nu_s = \frac{\sigma_s}{E_s \varepsilon_s} \]  
\[ (11) \]

\[ \nu_{sc} = \frac{\sigma_{sc}}{E_s \varepsilon_{sc}} \]  
\[ (12) \]

Where: \( \sigma_i, \sigma_s \) and \( \sigma_{sc} \) - stress in i-layer of concrete, in steels in tension zone and compression zone respectively.

Strain of materials are determined from the equilibrium of cross section as follows:

\[ \varepsilon_{bi} = \frac{M Z_{bi}}{D_{11}} \]  
\[ (13) \]

\[ \varepsilon_g = \frac{M Z_g}{D_{11}} \]  
\[ (14) \]

\[ \varepsilon_s = \frac{M Z_s}{D_{11}} \]  
\[ (15) \]

\[ \varepsilon_{sc} = \frac{M Z_{sc}}{D_{11}} \]  
\[ (16) \]

Coordinate of neutral axis of cross section \( y_0 \):

\[ y_0 = \frac{\sum_{i=1,...} A_i y_i E_i \sigma_i \varepsilon_i + A_i y_i E_i \sigma_i \varepsilon_i + A_i y_i E_i \sigma_i \varepsilon_i + A_i y_i E_i \sigma_i \varepsilon_i}{D_{33}} \]  
\[ (17) \]

Rigidity characteristic of cross section \( D_{33} \) is defined as follows:

\[ D_{33} = \sum_{i=1,...} A_i E_i \sigma_i \varepsilon_i + A_i E_i \sigma_i \varepsilon_i + A_i E_i \sigma_i \varepsilon_i + A_i E_i \sigma_i \varepsilon_i \]  
\[ (18) \]

Steel yielding condition:
$e_y = e_{s,y}$

Where: $e_{s,y} = \sigma_y/E_y$ - strain of steel at yield point.

The reinforced concrete beam collapses when one of the following conditions is met:

$$|e_{b,max}| \geq e_{b,ult}$$  \hspace{1cm} (20)

$$e_{s,max} \geq e_{s,ult}$$  \hspace{1cm} (21)

$$e_{g,max} \geq e_{g,ult}$$  \hspace{1cm} (22)

Where: $e_{b,max}$ - strain of the most compressive concrete fibre in the normal section due to external load; $e_{s,max}$ and $e_{g,max}$ - strain of the most tensile steel and GFRP rebars respectively in the normal section due to external load; $\sigma_{y} = 0.0035$ (Figure 15a) - ultimate strain of compressive concrete assumed in accordance to SP 63.13330.2018 [23]; $\varepsilon_{s,ult} = 0.2$ - ultimate strain of steel reinforcement (Figure 15b); $\varepsilon_{g,ult}$ - ultimate strains of GFRP reinforcement according to tensile test result [14]: $\varepsilon_{g,ult} = R_y/E_y$ (Figure 4).

Calculation is performed by iteration method. At first iteration, assuming an initial value of moment $M_{i}=\Delta M$. At this step, assume that the materials exhibit elastic behavior (i.e. $\varepsilon_{s,y} = \varepsilon_{g,max} = 1$), we determine the rigidity characteristic $D_{ij}$ (18), neutral axis position $y_{0}$ (17) and the rigidity characteristic $D_{11}$ (9). Next, we define strains of materials $\varepsilon_{bi}$ (13), $\varepsilon_{g}$ (14), $\varepsilon_{s}$ (15) and $\varepsilon_{e}$ (16). Stresses in concrete layers, tensile and compressive steel bars, GFRP bars ($\sigma_{g,y}$, $\sigma_{s}$, $\sigma_{g}$) are determined by stress-strains diagrams on Figure 12 and Figure 4 according to received strain values of the materials. After that we define the elastic coefficient $v_{bi}$, $v_{s}$ and $v_{sc}$ by (10), (11) and (12) for the second iteration.

At the end of first step, from the found stress values of materials the actual moment of this step is calculated:

$$M_{1} = \sum_{i=1}^{n} A_{bi} \sigma_{bi} Z_{bi} + A_{s} \sigma_{s} Z_{s} + A_{sc} \sigma_{sc} Z_{sc} + A_{g} \sigma_{g} Z_{g}$$  \hspace{1cm} (22)

In each subsequent step, the calculation is performed with initial moment $M_{i}=M_{i-1}+\Delta M$. At each step, the strain values of materials are controlled according to (19), (20), (21) and (22). At $n$-step the values of strain in tensile steel bar reaches the yielding strain by condition $e_{s} \geq e_{s,ult}$ for the first time, the steel yielding moment $M_{i}$ is taken equal the moment value at this step. Iterative process is continued until one of conditions (20), (21) and (22) is met. Here, we determine the failure mode and flexural capacity $M_{n}$ of the section, which is taken from the value of moment in the previous step. The calculation results of steel yielding moment and flexural capacity of the tested beams and comparison of the experimental and theoretical values of these moments are displayed in Table 4. The ratios of $M_{n,thor}/M_{n,exp}$ range from 0.88 to 1.11 and the ratios of $M_{1,thor}/M_{1,exp}$ vary between 0.86 and 0.95. Overall, the predicted values obtained from the current analysis are in good agreement with the experimental results for both hybrid GFRP/steel reinforced concrete beams and pure GFRP reinforced concrete beams.

To verify the proposed model, the experimental results of the hybrid GFRP/steel RC beams published in the literature [3, 11, 19] are compared to results obtained from proposed model as shown in Table 4. The average value of the ratio $M_{n,thor}/M_{n,exp}$ and $M_{1,thor}/M_{1,exp}$ are 0.93 and 1.01, respectively. These outcomes indicate that the proposed model is sufficiently reliable to estimate load carrying capacity.

| Reference | Beam ID | $h$, $mm$ | $b$, $mm$ | $A_{s}$, $cm^2$ | $A_{g}$, $cm^2$ | $A_{sc}$, $cm^2$ | Experimental results $M_{exp}$, kNm | Theoretical results $M_{thor}$, kNm | $\Delta_{1}$ | $\Delta_{2}$ |
|-----------|---------|------------|------------|---------------|---------------|---------------|----------------|----------------|----------|----------|
| This Study | 2G10-1S0 | 151        | 253        | 1.23         | 0.57          | -             | 23.0           | 25.6           | -        | 1.11     |
|           | 2G10-2S10 | 152       | 254        | 1.23         | 1.57          | 0.57          | 15.7           | 14.9           | 34.4     | 0.95     |
|           | 2G10-2S12 | 155       | 252        | 1.23         | 2.26          | 0.57          | 21.1           | 19.8           | 35.5     | 0.94     |
|           | 2G10-2S14 | 150       | 253        | 1.23         | 3.08          | 0.57          | 26.2           | 24.6           | 38.0     | 0.94     |
|           | 2G12-1S0 | 151        | 249        | 1.87         | 0.57          | -             | 28.6           | 25.2           | 34.4     | 0.97     |
|           | 2G12-2S10 | 150       | 250        | 1.87         | 1.57          | 0.57          | 17.6           | 16.2           | 41.3     | 0.92     |
|           | 2G12-2S12 | 155       | 250        | 1.87         | 2.26          | 0.57          | 24.9           | 21.5           | 45.6     | 0.87     |
|           | 2G12-2S14 | 151       | 248        | 1.87         | 3.08          | 0.57          | 29.2           | 25.2           | 45.0     | 0.86     |
2G14-S0  150  256  2.65  -  0.57  -  45.1  -  39.7  -  0.88
2G14-S10  151  251  2.65  1.57  0.57  19.8  48.4  17.6  43.6  0.89  0.90
2G14-S12  148  250  2.65  2.26  0.57  24.6  45.8  23.3  48.0  0.95  1.05
2G14-S14  153  255  2.65  3.08  0.57  32.3  56.5  27.8  49.5  0.86  0.88
3G14-S0  148  255  3.97  -  0.57  -  52.4  -  50.4  -  0.96
3G14-S10  152  254  3.97  1.57  0.57  23.4  49.4  20.8  49.6  0.89  1.00
3G14-S12  153  254  3.97  2.26  0.57  28.1  53.1  25.6  52.8  0.91  1.00
3G14-S14  155  255  3.97  3.08  0.57  34.5  56.7  32.4  60.0  0.94  1.06

| A1  | 150  | 200  | 0.883 | 1.00  | 1.01  | - | 25.14  | - | 22.4  | - | 0.89 |
| A2  | 150  | 200  | 1.57  | 1.00  | 1.01  | - | 28.41  | - | 27.9  | - | 0.98 |
| A3  | 150  | 200  | 2.36  | 2.26  | 1.01  | - | 35.55  | - | 35.8  | - | 1.01 |
| B2  | 150  | 200  | 0.88  | -  | 1.01  | - | 20.21  | - | 19.3  | - | 0.95 |
| C1  | 150  | 200  | 0.88  | 1.00  | 1.01  | - | 25.14  | - | 24.0  | - | 0.95 |

| G0.3-MD1.0-A90 | 280  | 380  | 2.84  | 9.82  | 1.01  | 101.0  | 147.0  | 111.0  | 168.0  | 1.10  | 1.14 |
| G1.0-T0.7-A90 | 280  | 380  | 9.82  | 6.28  | 1.01  | 161.0  | 261.0  | 152.0  | 232.0  | 0.94  | 0.89 |
| G0.6-T1.0-A90 | 280  | 380  | 5.67  | 9.82  | 1.01  | 178.5  | 229.0  | 184.0  | 240.0  | 1.03  | 1.05 |

| 2G12-S12 | 180  | 300  | 2.26  | 2.26  | 1.01  | 39.1  | 57.5  | 34.2  | 62.7  | 0.87  | 1.09 |
| 2G16-S12 | 180  | 300  | 4.02  | 2.26  | 1.01  | 44.3  | 63.3  | 39.6  | 79.8  | 0.89  | 1.26 |
| 2G12-1S16 | 180  | 300  | 2.26  | 2.01  | 1.01  | 38.2  | 56.4  | 32.4  | 61.2  | 0.85  | 1.09 |
| 2G16-1S16 | 180  | 300  | 4.02  | 2.01  | 1.01  | 45.4  | 66.7  | 38.4  | 75.6  | 0.85  | 1.13 |
| 2G12-2S12 (D) | 180  | 300  | 2.26  | 2.26  | 1.01  | 37.7  | 53.8  | 31.2  | 58.8  | 0.83  | 1.09 |

Note: $A_1=M_{u,exp}/M_{u,thor}$, $A_2=M_{u,exp}/M_{u,thor}$, The values $M_i$ are not reported by authors, these values are determined by load-deflection curves in these literatures.

5. Conclusions

This paper analyzes the influence of reinforcement ratios as well as configurations to the flexural behavior of hybrid GFRP/steel RC beams and GFRP RC beams. The research is carried out with a wide range of steel reinforcement ratio $\mu_s$, GFRP reinforcement ratio $\mu_g$ and ratio of reinforcements $\rho_g$. From the study results, the following conclusions can be drawn:

- There are four stages of behavior and six failure modes of hybrid GFRP/steel RC beams depending on the longitudinal reinforcement ratios. The findings make a contribution to the existing literature that presented only three stages of behavior and four failure modes of hybrid FRP/steel RC beams;
- The crack patterns, stiffness, ductility and toughness are considerably influenced by the percentage of longitudinal reinforcements $\mu_s, \mu_g$ and the ratio of longitudinal reinforcements $\rho_g$;
- It is better to use the energy ductility index to access the ductility of hybrid GFRP/steel RC beams. Using other approaches may lead to significant deviations. Based on the energy ductility index and energy ratio, the failure of hybrid GFRP/steel RC beams can be classified as ductile, less ductile and brittle.
- The presence of GFRP reinforcement delays the steel yielding of hybrid RC beams, and the relationship between GFRP reinforcement ratio and steel yielding load is linear. At the same total reinforcement ratio, the load-carrying capacity of hybrid RC beams is improved when increasing the ratio of reinforcement, $\rho_g$.
- The proposed analytical model using non-linear deformation models of materials can properly predict the steel yielding moment and ultimate flexural capacity of hybrid GFRP/steel RC beams. This model can also be used to determine the bearing capacity of GFRP RC beams.

Future research can consider the effect of concrete mixes and reinforcement arrangement on flexural behavior of the hybrid RC beams. In addition, the plastic hinge zones in concrete beams with hybrid bars could also be studied.

6. Conflicts of Interest

The authors declare no conflict of interest.
7. References

[1] Leung, H.Y., and R.V. Balendran. “Flexural Behaviour of Concrete Beams Internally Reinforced with GFRP Rods and Steel Rebars.” Structural Survey 21, no. 4 (October 2003): 146–157. doi:10.1108/02630800310507159.

[2] Qu, Wenjun, Xiaoliang Zhang, and Haiqun Huang. “Flexural Behavior of Concrete Beams Reinforced with Hybrid (GFRP and Steel) Bars.” Journal of Composites for Construction 13, no. 5 (October 2009): 350–359. doi:10.1061/(asce)cc.1943-5614.0000035.

[3] Lau, Denvid, and Hoat Joen Pam. “Experimental Study of Hybrid FRP Reinforced Concrete Beams.” Engineering Structures 32, no. 12 (December 2010): 3857–3865. doi:10.1016/j.engstruct.2010.08.028.

[4] Mustafa, Suzan A.A., and Hilal A. Hassan. “Behavior of Concrete Beams Reinforced with Hybrid Steel and FRP Composites.” HBRC Journal 14, no. 3 (December 2017): 300–308. doi:10.1108/hbrcj.2017.01.001.

[5] Pang, Lei, Wenjun Qu, Peng Zhu, and Jiajing Xu. “Design Propositions for Hybrid FRP-Steel Reinforced Concrete Beams.” Journal of Composites for Construction 20, no. 4 (August 2016): 04015086. doi:10.1061/(asce)cc.1943-5614.0000654.

[6] Xingyu, Gu, Dai Yiqing, and Jiang Jiwang. “Flexural Behavior Investigation of Steel-GFRP Hybrid Reinforced Concrete Beams Based on Experimental and Numerical Methods.” Engineering Structures 206 (March 2020): 110117. doi:10.1016/j.engstruct.2019.110117.

[7] Bui, Linh Van Hong, Boonchai Stitmannaithum, and Tamon Ueda. “Ductility of Concrete Beams Reinforced with Both Fiber-Reinforced Polymer and Steel Tension Bars.” Journal of Advanced Concrete Technology 16, no. 11 (November 14, 2018): 531–548. doi:10.3151/jact.16.531.

[8] Maranan, G.B., A.C. Manalo, B. Benmokrane, W. Karunasena, P. Mendis, and T.Q. Nguyen. “Flexural Behavior of Geopolymer-Concrete Beams Longitudinally Reinforced with GFRP and Steel Hybrid Reinforcements.” Engineering Structures 182 (March 2019): 141–152. doi:10.1016/j.engstruct.2018.12.073.

[9] Kara, Ilker Fatih, Ashraf F. Ashour, and Mehmet Alpaslan Köroğlu. “Flexural Behavior of Hybrid FRP/steel Reinforced Concrete Beams.” Composite Structures 129 (October 2015): 111–121. doi:10.1016/j.compstruc.2015.03.073.

[10] Kim, Seongeun, and Seunghun Kim. “Flexural Behavior of Concrete Beams with Steel Bar and FRP Reinforcement.” Journal of Asian Architecture and Building Engineering 18, no. 2 (March 4, 2019): 89–97. doi:10.1080/13467581.2019.1596814.

[11] Ruan, Xiangjie, Chunhua Lu, Ke Xu, Guanyu Xuan, and Mingzhi Ni. “Flexural Behavior and Serviceability of Concrete Beams Hybrid-Reinforced with GFRP Bars and Steel Bars.” Composite Structures 235 (March 2020): 111772. doi:10.1016/j.compstruct.2019.111772.

[12] Sun, Zeyang, Linchen Fu, De-Cheng Feng, Apete R. Vatuloka, Yang Wei, and Gang Wu. “Experimental Study on the Flexural Behavior of Concrete Beams Reinforced with Bundled Hybrid steel/FRP Bars.” Engineering Structures 197 (October 2019): 109443. doi:10.1016/j.engstruct.2019.109443.

[13] ACI Committee. "Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer (FRP) Bars (ACI 440. 1R-15).” Farmington Hills, Michigan: American Concrete Institute (2015).

[14] Lucier, G. “Tension Tests of GFRP Bars (Prepared for: Fiber reinfor polymer Viet Nam).” North Carolina State University (2016): p. 7.

[15] Baikov, V.N. and E.E. Sigalov. “Reinforced concrete structures. General Course. Textbook for Higher Institutes of Learning, 5th Edition” (1991).

[16] Jia, B., S. Liu, X. Liu, and R. Wang. “Flexural Capacity Calculation of Hybrid Bar Reinforced Concrete Beams.” Materials Research Innovations 18, no. sup2 (May 2014): S2–836–S2–840. doi:10.1179/1432891714z.00000000498.

[17] Ge, Wenjie et al. “Flexural behavior of concrete beam with hybrid reinforcement of FRP bars and steel bars.” Journal of Southeast University. Natural Science Edition 42.1 (2012): 114-119.

[18] Ge, Wenjie, Jiwen Zhang, Dafu Cao, and Yongming Tu. “Flexural Behaviors of Hybrid Concrete Beams Reinforced with BFRP Bars and Steel Bars.” Construction and Building Materials 87 (July 2015): 28–37. doi:10.1016/j.conbuildmat.2015.03.113.

[19] Aiello, Maria Antonietta, and Luciano Ombres. “Structural Performances of Concrete Beams with Hybrid (Fiber-Reinforced Polymer-Steel) Reinforcements.” Journal of Composites for Construction 6, no. 2 (May 2002): 133–140. doi:10.1061/(asce)1090-0268(2002)6:2(133).

[20] Grace, N. F., A. K. Soliman, G. Abdel-Sayed, and K. R. Saleh. “Behavior and Ductility of Simple and Continuous FRP Reinforced Beams.” Journal of Composites for Construction 2, no. 4 (November 1998): 186–194. doi:10.1061/(asce)1090-0268(1998)2:4(186).
[21] Naaman, A. and S. Jeong. “Structural ductility of concrete beams prestressed with FRP tendons.” Non-Metallic (FRP) Reinforcement for Concrete Structures: Proceedings of the Second International RILEM Symposium, Vol. 29 (23-25, August 1995).

[22] Polakowski, Natalis Horace, and Edward Joseph Ripling. "Strength and structure of engineering materials." (1966).

[23] NIIZHB named after A. A. Gvozdev. “Concrete and reinforced concrete structures. General provisions (SP 63.13330:2018).” Federal Registry of National Building Codes & Standards (2019).
Salt Gradation Analysis for Winter Road Maintenance

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Abstract

This research investigates the salt gradation specifications adopted by different provincial or state highway departments in Canada and the US for winter road maintenance operations. To understand the type of used salt, its quantity, grain size distribution, application method and the level of satisfaction of the user, a questionnaire was prepared and sent to selected provincial/state highway departments in Canada and the US. The survey-based comparative analysis performed on the salt gradation in different jurisdictions showed that the salt gradation does not always fit in ASTM (American Society of the International Association for Testing and Materials) and BS (British Standard) standard curves. However, it was found that the gradation of coarse and fine salt used by most Canadian provinces follows ASTM I and the Finnish standards, respectively. Although the majority of jurisdictions surveyed in this study have specific requirements for gradation of the salt used in their winter maintenance operations, no laboratory tests or field trials have been conducted to investigate the effectiveness of a particular salt gradation for road winter maintenance operations. It was also found that salt gradation standards are compromised due to factors such as local availability of the material, purity of the available material, ease of material handling, ease of application, and the preference of private contractors for certain materials.

Keywords: Winter Road Maintenance; Salt Gradation Specification; Coarse Salt Gradation; Fine Salt Gradation; Comparative Study.

1. Introduction

Winter road maintenance is a critical task of highway maintenance agencies in countries with extreme winter climatic conditions. Solid de-icing road salts are one of the most commonly adopted pavement treatment options by highway maintenance agencies (agencies) during winters [1]. Application of solid de-icing salts, to improve adhesion and smooth flow on ice-covered roadways, is an effective technique for mitigating snow from pavement surfaces, having favorable impact on vehicular floe, public safety, and substantial reduction in travel costs [2]. Different gradations of road salts can be utilized when performing anti-icing or de-icing activities. Most winter maintenance agencies including state or provincial highway departments generally use pre-defined salt gradation specifications that suit their operational requirements. Having pre-defined salt gradation specifications ensures that the de-icing material procurement process is facilitated. This also helps the maintenance agencies adhere to consistent operating and safety standards.

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However, depending on factors like material availability and budgetary constraints (material cost), agencies may sometimes compromise on the salt gradation specifications. Currently, winter maintenance agencies select de-icing salts based on two main factors which are cost of salt product and salt ability to perform under different road climatic conditions. Due to limited available budget for winter maintenance by public road agencies, material cost becomes the most significant factor selection of de-icing product. With that regard, sodium chloride (NaCl) due to its very low cost and high abundance is by the most commonly used salt material for de-icing during snow periods [3, 4]. Although the effectiveness of sodium chloride salt on snow covered roadways depends on various factors, the sodium chloride salt has an excellent performance in terms of de-icing ability at temperatures up to -9 °C to -12 °C. Furthermore recent field investigations have showed that under specific guidelines, sodium chloride salt could be used for de-icing at much lower temperature down to -20 °C [5-7].

The highway agencies understand the distinct benefits of both fine and coarse graded materials when used for ice control activities. Also, the percentage purity and particle size of salt material are principally essential considerations in selection and application of salt products for de-icing applications. Precisely, sodium chloride particle size strongly influences the effectiveness and activation time as well as the snow melting time duration on snow covered roadways [8].

Generally, fine salt particles are believed to be more effective anti-icing agents as compared to the coarse salt particles. The higher “specific surface” of the fine salt particles allows easier mixing with snow and ice particles in comparison with coarse particles. However, it has been found that the addition of coarse salt particles to road salt pile may improve the overall performance of salt as an anti-icing agent. There is an increased possibility of the finer salt grains getting blown away easily from the pavement surface due to turbulent air currents created by significantly high volumes and high speeds of traffic movement on the highways in combination with strong wind gusts during winter. Coarse salt grains have larger grain size and are heavier compared to fine salt grains. These properties of the coarse salt allow the particles to remain on the pavement surface for a prolonged period of time. It has been observed that overall wastage of coarse salt is less as compared to fine salt due to lower chances of being blowing away from the pavement surface as well as less slippage through the narrow gaps in the truck bodies used to carry salt [2].

The key objective of utilizing the salt as a de-icing agent is to destabilize the snow or ice bond on the pavement surface. This is done in steps including melting of the snow or ice pack and formation of brine on the pavement surface. Once the bonds are broken, the snow and ice float on the brine solution. The movement of traffic facilitates further breaking the ice chunks and creates a slush that can be mechanically plowed away. Fine salt particles are known to accelerate the snow melting process and formation of brine on the pavement surface. Fine salt particles, therefore, significantly enhance the process of melting and brine formation. However, when applied on thicker snow or ice packs, the finer particles may not penetrate deeper into the snow and may not reach the pavement. Moreover, at lower temperatures refreezing may occur as fine particles may lose their effectiveness by diluting quickly. Theoretically, the addition of coarse particles may enhance the performance of road salt as a de-icing agent as coarse particles will stay on the icy surface for a longer period and allow slower continuous reaction of salt with the ice pack. Ideally a well-blended mixture of coarse and fine salt particles would significantly improve the effectiveness of the road salt both as an anti-icing or de-icing agent. Therefore, suitable salt gradation standards need to be established and practiced by winter maintenance agencies in order to maximize the effectiveness of road salt during de-icing or anti-icing activities [1, 9].

2. Objectives and Scope

This research aims to accomplish the following objectives:

- To review the theoretical and practical aspects of road salt gradation for road winter maintenance operations;
- To investigate the salt gradation standards set by international standardization organizations such as ASTM (American Society of the International Association for Testing and Materials) and BS (British Standard), and national standards in countries such as Finland and Sweden;
- To investigate the salt gradation specifications adopted by different highway departments in Canada and the US and compare them with international standards to provide an overview of the graduation standards adopted by agencies facing similar challenges during winter months.

To understand how highway departments in Canada and the US manage the salt gradation of de-icing materials, an outreach program was planned that included a questionnaire survey, telephonic interview, and in-person meetings and interviews. The collected data were analyzed and compared with available standards for salt particle size distributions set by international organizations. Figure 1 shows the research methodology.
Figure 1. Research Methodology

3. Background and Previous Studies

3.1. Types of Processed Salt Used in Winter Maintenance

Recently, winter maintenance agencies select de-icing salts based on two main factors which are cost of salt product and salt ability to perform under different road climatic conditions. Sodium chloride (\(\text{NaCl}\)) due to its very low cost and high abundance is by the most commonly used salt material for de-icing during snow periods [2].

Rock salt or Sodium chloride is the most commonly used ice-control chemical for application on roads and highway in the US and Canada. It is generally produced in three forms namely rock salt, solar salt and evaporated or solution (or vacuum) salt. While the rock salt is mined utilizing conventional hard rock mining equipment and techniques, solar salt is produced by the evaporation of sea water and the evaporated (or solution or vacuum salt) is made by vacuuming and drying the solution created after injecting water into underground salt deposits. Typical characteristics of these processed salts are highlighted in Table 1 [10].

| Processed salt | Grain size | Moisture content | Impurities |
|---------------|------------|-----------------|------------|
| Solar salt    | Irregular  | High to very high | Average    |
| Rock salt     | 0-5 mm     | Low, <0.1%       | 1-4%       |
| Vacuum salt   | Around 0.6 mm | Average, around 2.5% | Low       |

While solar salt is produced in several western states of the USA, some quantity is imported into the eastern US [10]. Canada is a significant consumer of rock salt. It has a vast geographical territory with several known rock salt deposits. In addition, there is a huge potential for new discoveries within Canada. There are only a small number of companies that have currently exploited the vast known reserves in Canada, which are also the major players in Canada’s road salt industry including extraction and processing of rock salt. Some of these companies also export rock salt. Canada also imports salt from the USA. Most of the salt is utilized for ice-control applications on roads during winter, production of a vast range of chemicals, and domestic consumption (e.g., table, food-grade, livestock feed) [11].

Major Canadian salt deposits are found in the provinces of Nova Scotia, New Brunswick, Quebec, Ontario, Manitoba, Saskatchewan, and Alberta. The largest rock salt deposits are in western Canada, followed by Ontario and the Atlantic provinces. In western Canada, the rock salt beds start from the Northwest Territories and cover vast regions in Alberta, Saskatchewan, and extend up to Manitoba. Many salt deposits were discovered while these regions were explored for oil, gas and other minerals like potash. This huge salt deposit contains over one million billion tons of salt. The salt deposit is approximately 122 m in thickness and covers an area of approximately 390,000 km2 [11].

Review of the 2009 data indicates that in Canada, 93.0% of the salt was mined as rock salt from major rock salt mines located in Ontario, Quebec, and New Brunswick, 6.0% of the production was using fine vacuum method from vacuum pan salt refineries in Alberta, Saskatchewan, Ontario, New Brunswick, and Nova Scotia, and 1.0% was produced as brine or salt recovered as part of chemical operations. The majority of the salt production was rock salt that was used for highway de-icing activity [11].
3.2. Application Types and Significance of Gradation

Solid ice-control road salts are the most effective anti-icing agents when applied before ice forms on the pavement. Other case studies show that solid chemicals (e.g., salt) could be effectively used when applied at traffic speeds under about 50 km/h and traffic volumes under 100 vehicles per hour as a pre-treatment [12]. Some studies indicate that using salt with “coarser” gradation or particle size distribution is very appropriate for de-icing operations [13]. The reason is that large salt particles melt through the surface and cause ice melt as well. This will continue until the existing bond between pavement surface and ice is broken and at this stage it would be possible to remove the snow or ice mechanically. However, fine graded salt may be frozen or washed out of the pavement surface, hence using fine graded salt for de-icing operations is generally not cost effective compared to the use of coarse graded salt [12].

3.2.1. Fine Grained Salt

Fine graded salt has a high dilution potential and may blow off the road surface because of moving traffic and winds. Hence, it is not suitable for de-icing operations [12].

Comparing to coarse grain salt, fine salt dilutes much faster and as a result, it should be re-applied more frequently and with a greater volume to have the same effectiveness as coarse salt at the same weather conditions. The reason is that it melts quickly and produces a wet pavement surface. However, this situation will not last for a long time since the wet surface will re-freeze again if the temperature is extremely low. The best application of fine graded salt is for the treatment of thin ice layers on the road surface and also for pre-treatment just before the daylight when pre-wetted [13].

As a preventive operation for black ice formation during the fall and spring when the pavement temperature is around 0° to -3°C, fine grained salt may be applied as solution or liquid salt. In this case liquid salt should be applied up to three hours before any precipitation happens or road surface becomes frozen. The advantages of using liquid salt are: decreasing the total amount of used salt, its instant melting effect, and not splashing against car windshields [12].

In situations where the pavement temperatures are lower than -5°C or application on compacted ice is considered, liquid salt is not recommended. In addition, although application of liquid salt is not recommended in freezing rain conditions, occasionally it has been used with increased application rate up to 40 to 60 g/m². For roads with low traffic volume (less than 2,000 ADT) the effectiveness of liquid salt is lower [12].

The major disadvantages of using liquid salt is increasing the operators workload since after application; they need to observe the road surface continuously to make sure it is not frozen. Also, the salt brine is very corrosive and may damage the spreading equipment and its electrical connections quicker compared with coarse grain salt [12].

3.2.2. Coarse Grained Salt

The success of salt spreading has a strong relationship with its grain size distribution. If the salt grain is fine, it makes the ice melt fast on the surface; however, the resulting brine refreezes rapidly and will not penetrate to the surface adequately. On the other hand, large salt grains (3 to 5 mm) slowly penetrate into the pavement surface, stay longer on the road but need more time to work since they dissolve at a slower rate. Another problem with coarse grain salt is that large grains may bounce back to the road shoulder during the spreading operations [12].

The commonly used salt for anti-icing or de-icing operations is sodium chloride. A mix of solid sodium chloride and solid calcium chloride or calcium chloride solely has also been used by some agencies. The grain size distribution of the salt that is used for anti-icing treatment is very similar to what is designed for de-icing operations, without pre-wetting conditions [10].

Based on the research conducted in various European countries, the ideal salt grain size used in winter maintenance ranges from 0.16 to 3 mm, with a low percentage (maximum 5%) ranging below 0.16 mm or between 3 and 5 mm and not greater than 5 mm. However, some countries have regulation to use large grain salts (at least 5%) because of its deep-acting effect [12].

In Canada, a study conducted by researchers at the University of Waterloo [14] concluded that salt gradation has great impact on the performance of the de-icing salt. A comparative study was conducted between coarse road salt with average particle size of almost 6.5 mm (the majority of the material being in the range of 4 to 9 mm); and fine road salt with average particle size of 4.3 mm (the majority of the material within the range of 1.6 to 7 mm); and another road salt (referred to as UW salt) with average particle size of 2.5 mm (the majority of the material <5 mm). The performance comparison of different salt gradations was conducted by creating a salting rate model to calculate the appropriate salting rate considering various conditions such as temperature, salt weight concentration, and the time taken for the pavement to become 80% or more snow and ice free (bare pavement regain time). The results showed that the fine graded road salt was 20% more effective than coarse road salt, and the UW road salt was approximately 60% more effective than coarse road salt as the coarse salt can be easily dispersed by vehicular traffic. It is worth
mentioning that this study was conducted on parking lots, so the results may not be similar on highways due to the impact of moving traffic.

Another study at the University of Nebraska [15] investigated the impact of salt gradation on refreeze time. It was shown that there is a strong relationship between salt grain size distribution and its refreeze time. Samples with gradations smaller than 0.422 mm (sieve #40) were refreezing very fast, while samples with larger grain size salts of about 2.38 mm (#8 sieve) were refreezing after two hours.

Table 2 summarizes the advantages and disadvantages of using different gradations of salt for de-icing and anti-icing applications.

Table 2. Typical characteristics of processed salts

| Agent            | Application | Advantages                                                                 | Disadvantages                                                                 |
|------------------|-------------|----------------------------------------------------------------------------|-------------------------------------------------------------------------------|
| Coarse Salt      | De-icing    | - Provides increased traction                                             | - Can be rolled and displaced towards the road shoulders due to traffic movement |
|                   |             | - Can penetrate the snow lifts quicker due to its weight in comparison to fine salt | - May not be effective below certain freezing points (depending on salt type: -6° to -25°C approx.) |
|                   |             | - Can be mixed with sand                                                   | - It has a dilution potential                                                  |
|                   |             | - Needs longer time to leach to the nearby soils as it takes time to dissolve compared to fine and liquid salt | - Takes time to dissolve, melt and start working as it comes in relatively larger grains compare to fine salt |
|                   |             | - Can be used either dry or pre-wetted                                     | - Overuse of salt can impact the surrounding environment and infrastructure     |
|                   | Anti-icing  | - Pre-wetted salt can remain on the paved surface.                         | - Not appropriate because of long dilution time                               |
|                   |             | - A suitable gradation is appropriate in the absence of pre-wetting.       | - Cannot stay on the paved surface due to wind and traffic                     |
|                   |             |                                                                              | - Needs minimum moisture to trigger the effectiveness of the dry chemical     |
| Fine Salt         | De-icing    | - Needs short time to melt and start working                              | - Can be blown away during spreading                                           |
|                   |             | - Can easily be mixed with liquids                                         | - May not be effective below certain freezing points (depending on salt type: -6° to -25°C) |
|                   |             | - Can be mixed with sand                                                   | - It has a dilution potential                                                  |
|                   |             | - Can be used either dry or pre-wetted                                     | - It takes time to dissolve and penetrate snow/ice lifts                      |
|                   |             | - Needs longer time to leach to the nearby soils compared to liquid salt    | - Overuse of salt can impact the surrounding environment and infrastructure     |
| Anti-icing        |             | - Its use will decrease the time for a solution compared to coarse salt     | - Due to their larger surface area, they need to be heavily wet with a liquid to stand up to traffic and wind fairly well. |
|                   |             |                                                                              | - Spreading truck speed should not be so high                                  |
| Liquid Salt/brines/other liquid chemicals | De-icing | - May be used in limited situations for de-icing if the treatment is immediately followed by an application of solid chemicals. | - Since liquid ice control chemicals are mostly water, they are already fairly well diluted. They are not well suited to de-icing operations as they have little ability to penetrate thick snow ice. |
|                   | Anti-icing  | Can work immediately after spraying                                        | - Cannot be mixed with sand as it will be difficult to spry                   |
|                   |             | - Easy to handle/control and spray                                          | - Ineffectiveness at pavement temperatures below -5°C                        |
|                   |             | - It is most effective when used for black ice and during preventive operations | - Not recommended during either a freezing rain or sleet storm because of the large quantity needed to retain an effective concentration |
|                   |             | - Higher friction and better pavement conditions early in a storm           | - Overuse can impact the surrounding environment and infrastructure           |
|                   |             |                                                                              | - Needs less time to leach down to nearby soils compared to coarse and fine salt. |

3.3. International Salt Gradation Standards

The American Society of the International Association for Testing and Materials (ASTM) Specifications of natural sodium chloride from rock salt deposits or manmade produced salt by evaporation, solar or other methods are covered by ASTM D632-12 [16]. This standard recognizes two types and grades as below:

- Type I: used primarily as a pavement de-icer or in aggregate stabilization;
- Type II: used in aggregate stabilization or for purposes other than de-icing.

The grading of Type I sodium chloride shall conform to the following requirements for particle size distribution:
Grade 1: According to ASTM D632-12 [16], this grading is suitable for general applications and under most conditions is effective for increasing skid resistance and ice control.

Grade 2: this grading is typically being used and preferred for salt produced in the western US and states of the Rocky Mountains. The specifications are shown in Table 3.

| Table 3. ASTM D632 (2012), Type I, specification for salt gradation |
|---|---|---|---|---|---|
| Sieve size (mm) | 19.0 | 12.5 | 9.5 | 4.75 | 2.36 |
| | ⅞ in | ½ in. | ⅜ in. | No. 4 | No. 8 | No. 30 |
| Grade I (Mass % passing) | 100 | 100 | 95-100 | 20-90 | 10-60 | 0-15 |
| Grade II (Mass % passing) | 100 | ---- | ---- | 20-100 | 10-60 | 0-15 |

3.3.1. British Standard (BS)

The British Salt Standard [17] has specifications for two different grades i.e. rock salt and vacuum salt. According to the British road authority, compliance with the gradation will enhance the spreading ability of the coarse salt for treating snow and ice [12]. BS salt gradation standard requirements are shown in Table 4.

| Table 4. BS salt gradation requirements as a percentage passing [17] |
|---|---|---|---|---|---|
| Sieve size (mm) | 10 mm | 6.3 mm | 2.36 mm | 300 µm |
| Rock Salt | Mass % passing |
| Coarse (BS I) | 100 | 75 to 95 | 30 to 70 | 0 to 20 |
| Fine (BS II) | 100 | 100 | 30 to 80 | 0 to 20 |

| Sieve size (mm) | 10 mm | 6.3 mm | 1.18 mm | 150 µm |
|---|---|---|---|---|
| Vacuum Salt | Mass % passing |
| Coarse (BS III) | 100 | 100 | 0 to 80 | 0 to 10 |
| Fine (BS IV) | 100 | 100 | 100 | 0 to 30 |

3.3.2. Finnish Standard

In 1989, studies were conducted on the effect of rock salt gradation and the Finnish National Road Administration revised the specifications after that [9]. The outcome of the study showed that a finer gradation had better result with pre-wetting operations. In the original standard it was required that 100% of the material should pass through a 6-mm sieve, while the revised standard changed the 6-mm sieve to 5-mm. Table 5 shows the requirements for Finnish standard.

| Table 5. Finnish salt gradation specifications [10] |
|---|---|---|---|---|---|---|
| Sieve size (mm) | 5 | 4 | 3 | 2 | 1 | 0.5 |
| Mass % passing | 100 | 90-100 | 70-100 | 40-90 | 15-55 | 3-25 |

3.3.3. Swedish Standard

Based on the studies conducted by the Swedish Transport Administration, the salt’s origin does not have significant influence on its melting effect [10]. However, the grain size distribution is very important.

Fine-grained salt can prevent formation of thin ice layers on the pavements surface while coarse salt is more effective in slush producing during the snowfall. The Swedish gradation specifications are shown in Table 6. Figure 2 compares all mentioned standard gradations.

| Table 6. Swedish gradation specifications [10] |
|---|---|---|---|---|---|
| Sieve size (mm) | 3 | 2 | 0.5 | 0.16 |
| Mass % passing | 95-100 | 65-100 | 26-50 | 5-26 |
4. Data Collection and Results

4.1. Questionnaire Survey

A questionnaire-based survey was sent to selected provincial/state highway departments in Canada and the US. The questionnaire included the following questions:

1. What salt gradation does your organization use for de-icing operations in winter?

2. What is the flexibility of using salts with different gradations, i.e., do you always adhere to the above gradation for salt? If no, then under what situations a different gradation of salt would be used? Is there other gradation of salt used in the above situations?

3. Please rate your organization’s level of satisfaction with the current salt gradation on a scale of 1 to 5. In your opinion, what are the advantages and disadvantages of the current salt gradation used by your organization?

4. Has your organization ever conducted trials on salt gradation? If yes, please provide a brief description of the trials and findings.

5. How much salt do you consume in an average winter season (quantity in tons)?

4.2. Telephonic Interviews

Telephonic interviews were conducted with concerned officials from most of the above organizations to explain the purpose of the research and to provide any clarifications about the questionnaire.

4.3. Field Level Feedback (Anecdotal)

As part of the research, some field professionals from the winter maintenance industry were contacted including heavy duty truck mechanics from the highway departments, experienced contractor truck operators, and field representatives of the companies directly involved in calibrating the electronic controllers for de-icing trucks operated by highway departments or their contractors. The feedback received from these sources was used to crosscheck the survey findings.
4.4. Data Collection Results

Out of six responses received from the Canadian provinces (Nova Scotia Department of Transportation and Infrastructure; Government of Alberta, Ministry of Transportation; British Columbia Ministry of Transportation; Ministère des Transports du Québec; Ministry of Transportation of Ontario; Manitoba Infrastructure) and four responses from the Department of Transportation organizations in the US (Departments of Transportation (DOT) of North Dakota, District of Bismarck; Washington State; Michigan; New York State) it was found that most of these jurisdictions have well-defined salt gradation standards in place. However, a majority of them do not completely adhere to their gradation specifications at all times. The salt gradation standards are compromised due to factors such as local availability of the material, purity of the available material, ease of material handling, ease of application, and the preference of private contractors for certain materials. Some organizations chose a gradation specification that was suitable both for direct pavement application as well as in the form of a solution. Often the gradation specifications serve as high level guidelines for the operating and procurement departments as well as material suppliers. Organizations with multiple gradation specifications, like DOT New York State and the Ministry of Transportation of Ontario, tend to adhere closely to one of the specifications.

The study found that respondents generally considered coarse grade material more suitable for direct application on the pavement as it prevented the formation of a bond between the ice and pavement by staying longer on the pavement surface. The general handling of the coarse-graded material was considered convenient and safer. However, some organizations chose a finer grade material because of the ease of mixing in a solution without leaving any residue, as well as the ability to utilize it as a direct application product on the pavement. The organizations using fine grade material did mention the high amounts of material wastage due to leakage during transportation, particularly during direct application.

For Alberta, the major deciding factor was the local availability of the material. The British Columbia Ministry of Transportation does not have a preferred salt gradation and depend on their experienced contractors to choose the de-icing material.

Organizations like Nova Scotia Department of Transportation and Infrastructure and DOT North Dakota, District Bismarck prefer fine grade material as they also use the material to make brine solution in addition to using it for road salt application. The fine grade material produces less sediment in the brine solution and dissolves quickly compared to the coarse material. However, both organizations have received complaints regarding wastage due to the fine grade material leaking from the ice control truck bodies when applying straight salt on the roads.

The Ministère des Transports du Québec is quite satisfied with the coarser material as it stays on the ground for a longer period of time and it is easier to handle and apply.

Organizations like DOT Washington State, DOT New York State and the Ministry of Transportation of Ontario use different material gradation specifications based on the application. Fine grade material is used when applying the material in the form of a solution and coarse grade material is used when applying directly on the pavement.

4.5. Comparative Analysis of Salt gradations

Tables 7a to 7j reflect salt gradations used by different provinces in Canada and the US obtained from the survey results, and Table 8 shows the comparison of these gradations with available standards. For example, Figure 3 shows a comparison between ASTM I and II gradations with Alberta and Saskatchewan coarse salt gradations. Based on the analysis, results are summarized in Table 8 and the following conclusions are drawn:

- Saskatchewan’s coarse salt gradation is best fitted with ASTM II, while its mixed gradation fits in BS III gradation. The medium and fine gradations cannot be fitted into any standard curve; however, the medium gradation is close to the Finnish standard gradation;
- Alberta’s coarse salt gradation meets ASTM I and II and also BS III gradations. Fine gradation is almost similar to BS IV while medium gradation cannot be fitted into any standard gradations;
- Nova Scotia’s salt gradation is within ASTM I gradations;
- North Dakota’s salt gradation meets BS III and Finnish gradations;
- Quebec’s salt gradation also meets ASTM I specifications;
- Manitoba’s minimum limit of salt gradation meets both ASTM I and II standards, while the maximum limit is finer than the defined range in these two standards; the maximum limit is closer to ASTM II;
- New York State’s minimum limit of all salt gradations (Types A, B, C) match with the BS II standard while their maximum limits match with ASTM II. Overall, the gradations are close to ASTM II;
- Michigan DOT’s salt meets both ASTM I and II gradation standards;
- Washington State’s salt “Category A” gradation matches with ASTM II while the category B maximum limit is slightly finer than ASTM II; however, the difference seems not to be significant.

Table 7. Salt gradation of different jurisdictions

a. Saskatchewan’s salt gradation

| Sieve size (mm) | Coarse | Mixed | Medium | Fine |
|----------------|--------|-------|--------|------|
|                | min    | max   | min    | max  |
| 12             | 100    | 100   | 100    | 100  |
| 9              | 70     | 100   | 100    | 100  |
| 5              | 45     | 100   | 65     | 100  |
| 2              | 10     | 100   | 55     | 85   |
| 0.9            | 0      | 15    | 15     | 60   |
| 0.4            | 0      | 10    | 0      | 60   |
| 0.071          | 0      | 5     | 0      | 8    |

b. Alberta’s salt gradation

| Sieve size (mm) | Alberta I | Alberta II | Alberta III |
|-----------------|-----------|------------|-------------|
|                 | min   | max   | min   | max   | min   | max   | min   | max   |
| 10              |       | 100   | 100   |       | 100   |       | 100   |       |
| 5               | 100   | 100   | 67    | 100   | 55    | 90    |       |       |
| 2.5             | 98    | 100   | 40    | 85    | 5     | 60    |       |       |
| 1.25            | 85    | 100   | 25    | 67    | 0     | 25    |       |       |
| 0.63            | 35    | 100   | 15    | 45    | 0     | 8     |       |       |
| 0.315           | 0     | 50    | 0     | 17    | ---   | ---   | ---   | ---   |
| 0.16            | 0     | 8     | 0     | ---   | ---   | ---   |       |       |
| 0.089           | 0     | 3     | 0     | 3     | 0     | 3     |       |       |

c. Ontario’s salt gradation

| Sieve size (mm) | Coarse | Fine |
|-----------------|--------|------|
|                 | min    | max  | min | max |
| 9.5             | 100    | 100  |     |     |
| 4.75            | 30     | 100  | 100 | 100 |
| 2.36            | 5      | 65   | --- | --- |
| 1.18            | 0      | 30   | 35  | --- |
| 0.6             | 0      | 10   | --- | --- |

d. Nova Scotia’s salt gradation

| Sieve size (mm) | Mass percentage passing (%) |
|-----------------|-----------------------------|
|                 | Min | Max |
| 14              | 100 |     |
| 10              | 95  | 100 |
| 5               | 20  | 90  |
| 2.5             | 10  | 60  |
| 0.63            | 0   | 10  |

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### e. North Dakota DOT’s salt gradation

| Sieve size (mm) | Mass percentage passing (%) | Min  | Max   |
|-----------------|-----------------------------|------|-------|
| 19              |                             | 100  | 100   |
| 12.5            |                             | 100  | 100   |
| 9.5             |                             | 100  | 100   |
| 4.75            |                             | 95   | 100   |
| 2.36            |                             | 65   | 90    |
| 0.6             |                             | 5    | 25    |
| 0.3             |                             | 0    | 10    |

### f. Quebec’s salt gradation

| Sieve size (mm) | Mass percentage passing (%) | Min  | Max   |
|-----------------|-----------------------------|------|-------|
| 12.5            |                             | 100  | ---   |
| 10              |                             | 95   | 100   |
| 5               |                             | 20   | 90    |
| 2.5             |                             | 10   | 60    |
| 0.63            |                             | ---  | 11    |

### g. Manitoba’s salt gradation

| Sieve size (mm) | Mass percentage passing (%) | Min  | Max   |
|-----------------|-----------------------------|------|-------|
| 9.5             |                             | 95   | 100   |
| 4.75            |                             | 20   | 100   |
| 2               |                             | 10   | 85    |
| 0.425           |                             | 0    | 25    |

### h. New York State DOT’s salt gradation

| Sieve size (mm) | Mass percentage passing (%) | Type - A | Type - B | Type - C |
|-----------------|-----------------------------|---------|---------|---------|
|                 | Min | Max | Min | Max | Min | Max |
| 12.5            | 100 | 100 | 100 | 100 | 100 | 100 |
| 9.5             | 100 | 100 | 100 | 100 | 100 | 100 |
| 4.75            | 80  | 100 | 80  | 100 | 80  | 100 |
| 0.3             | 0   | 18  | 0   | 25  | 0   | 35  |
| 0.075           | 0   | 3   | 0   | 5   | 0   | 5   |

### i. Michigan DOT’s salt gradation

| Sieve size (mm) | Mass percentage passing (%) | Min  | Max   |
|-----------------|-----------------------------|------|-------|
| 12.5            |                             | 100  | 100   |
| 9.5             |                             | 95   | 100   |
| 4.75            |                             | ---  | 90    |
| 2.36            |                             | ---  | 60    |
| 0.6             |                             | ---  | 15    |
4.6. Comparison of Canadian Jurisdictions’ Coarse Salt Gradation to International Standards

According to Table 8, the six coarse salt gradations that are used in Canadian provinces are Saskatchewan, Alberta, and Ontario’s coarse salt gradation, and Nova Scotia, Quebec, and Manitoba’s general salt gradation. According to Table 8, some of the provinces’ coarse salt gradations, such as Saskatchewan, Alberta and Ontario, are compatible with more than one international standard. As presented in Figure 4a, among Canadian provinces with coarse salt gradation (6 provinces), 83% have gradation close to (or in range of) ASTM I, 67% to ASTM II, and 50% to BS III defined ranges for coarse salt gradation.

Table 8 shows four fine salt gradations used in surveyed Canadian provinces and US DOTs, which are Saskatchewan’s medium and fine salt gradation, Alberta and Ontario’s fine salt gradation, and North Dakota. According to Table 8, Saskatchewan’s medium salt gradation, North Dakota’s salt gradation, and Ontario’s fine salt gradation are close to the Finnish standard, while Alberta’s fine salt gradation is close to BS IV. As presented in Figure 4b, among the surveyed Canadian provinces and US DOTs with fine salt gradation (considering medium gradation for Saskatchewan as fine salt), 75% have gradation close to or in range of the Finnish standard and 25% to BS IV defined ranges for fine salt gradation.
Table 8. Comparison of salt gradations with available standards

| Standard          | ASTM I | ASTM II | BS I (Rock salt - coarse) | BS II (Rock salt - fine) | BS III (Vacuum & marine salt - coarse) | BS IV (Vacuum & marine salt - fine) | Swedish | Finnish |
|-------------------|--------|---------|---------------------------|--------------------------|----------------------------------------|--------------------------------------|---------|---------|
| Saskatchewan      |        |         |                           |                          |                                        |                                      |         |         |
| Coarse            | Almost I-R | I-R*    | Min: C* Max: F*          | Min: C Max: I-R          | Almost I-R                             | C                                    | C       | F       |
| Mixed             | Min: I-R Max: F | Min: I-R Max: F | Min: C Max: F          | Min: C Max: I-R          | Almost I-R                             | C                                    | Min: C Max: I-R | Min: C Max: I-R |
| Medium            | F       | F       | F                         | Min: I-R Max: F          | Min: C Max: I-R                        | C                                    | Min: C Max: I-R | Min: C Max: I-R |
| Fine              | F       | F       | F                         | F                        | Min: I-R Max: F                        | Min: C Max: I-R                      | Min: I-R Max: F | Min: I-R Max: F |
| Alberta           |        |         |                           |                          |                                        |                                      |         |         |
| Type III (Coarse) | Almost I-R | Almost I-R | Min: C Max: I-R          | Min: C Max: I-R          | Almost I-R                             | C                                    | C       | C       |
| Type II (Medium)  | F       | F       | F                         | Min: I-R Max: F          | Min: C Max: I-R                        | C                                    | Min: C Max: I-R | Min: C Max: I-R |
| Type I (Fine)     | F       | F       | F                         | F                        | Almost I-R                             | F                                    | F       |         |
| Ontario           |        |         |                           |                          |                                        |                                      |         |         |
| Coarse            | I-R     | I-R     | Min: C Max: I-R           | Almost I-R               | C                                      | C                                    | C       | C       |
| Fine              | Almost I-R | I-R     | F                         | F                        | C                                      | C                                    | C       | I-R     |
| Nova Scotia       |        |         |                           |                          |                                        |                                      |         |         |
|                  | Almost I-R | Min: C Max: I-R | C | C | C | C | C | C |
| North Dakota (NDDOT) | F | F | F | Min: I-R Max: F | Almost I-R | C | Min: C Max: I-R | I-R |
|                  | I-R     | Min: F Max: I-R | Min: C Max: I-R | C | C | C | C | C |
| Quebec            |        |         |                           |                          |                                        |                                      |         |         |
|                  | Min: I-R Max: F | Min: I-R Max: F | Min: C Max: F | Min: C Max: F | Min: C Max: F | C | C | Min: C Max: I-R |
| Manitoba          |        |         |                           |                          |                                        |                                      |         |         |
| Type A            | Min: F Max: I-R | Min: F Max: I-R | F | Min: I-R Max: C | Min: F Max: C | C | C | C |
| Type B            | Min: F Max: I-R | Min: F Max: I-R | F | Min: I-R Max: C | Min: F Max: C | C | C | C |
| Type C            | F       | Min: F Max: I-R | F | Min: I-R Max: C | Min: F Max: C | C | C | C |
| Michigan DOT      |        |         |                           |                          |                                        |                                      |         |         |
|                  | I-R | I-R | Almost I-R | C | C | C | C | C |
| Washington State  |        |         |                           |                          |                                        |                                      |         |         |
| Category 4A       | Min: C Max: F | I-R | Min: C Max: I-R | C | C | C | C | C |
| Category 4B       | Min: F Max: C | Almost I-R | Min: F Max: C | C | Min: F Max: C | C | C | C |

* F: Finer, C: Coarser, I-R: In Range
5. Conclusions

Highway departments all over North America understand the importance of salt gradation in the ice control process. Most of the highways agencies have a pre-defined salt gradation specifications suited to their operations. The availability of these specifications facilitates the de-icing material procurement process as well as helping to maintain operating and safety standards. However, the salt gradation standards are not adhered to at all times by all organizations due to local situations. This research investigated various salt gradation standards for winter maintenance operations set by international standardization organizations such as ASTM and BS, and national standards in countries such as Finland and Sweden. Further, the research compared the salt gradation standards implemented by some Canadian provinces and US DOTs with international standards. The information regarding salt gradations in Canadian provinces and US DOTs was retrieved through a questionnaire survey. The following summarizes the survey results and the conclusions of the conducted comparative study:

- The literature review showed that in general, finer salt grades created higher wastage (anecdotal), and coarser grades adhered better to road surfaces in direct applications.
- The questionnaire survey results revealed that, although it is essential to conduct laboratory investigation and field trials to confirm the effectiveness of a specific salt gradation for winter road maintenance operations, none of the jurisdictions surveyed in this study conducted such investigations.
- Most of the jurisdictions responding to the research survey have well-defined salt gradation standards for winter maintenance, drawn from ASTM, BS, and other standards bodies. However, in practice the standards are not always followed depending on the availability of the correct supply.
- Jurisdictions with defined multiple standards for differing road conditions tend to adhere closely to their standards as they have been more rigorously defined.
- The results of the comparative study at different jurisdictions revealed that:
  - Saskatchewan’s medium and fine salts do not fit in ASTM and BS standard curves for salt standards. Its coarse salts are within ASTM II standards. However, ASTM II salts are primarily used in non-de-icing conditions, for aggregate stabilization, and other uses.
  - Similar climatic jurisdictions such as Alberta, Ontario, Nova Scotia, and Quebec all use salts that fit readily into ASTM I specifications.
  - North Dakota uses a very coarse salt that meets British Standards. This could be because of supply and/or windy road conditions in which coarse salt has better adhesion.

Given the scarcity of empirical tests as reflected through survey results, additional laboratory testing and large-scale field trials are recommended to investigate the impact of salt gradation on the efficiency of winter road maintenance operations. Such laboratory tests and field trials would provide a better insight towards optimization of physical properties and application rates of various de-icing materials used in road winter maintenance operations under various environmental conditions.
It is worth mentioning that the concluded results are limited to the received responses from the only six Canadian provinces and four US DOTs. The availability of additional information from other jurisdictions would enhance the reliability and representativeness of the results.

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8. Conflicts of Interest
The authors declare no conflict of interest.

9. References
[1] Wu, Shujuan, Mulian Zheng, Wang Chen, Sitong Bi, Chongtao Wang, and Yifeng Li. “Salt-Dissolved Regularity of the Self-Ice-Melting Pavement Under Rainfall.” Construction and Building Materials 204 (April 2019): 371–383. doi:10.1016/j.conbuildmat.2019.01.129.
[2] Autelitano, Federico, Massimiliano Rinaldi, and Felice Giuliani. “Winter Highway Maintenance Strategies: Are All the Sodium Chloride Salts the Same?” Construction and Building Materials 226 (November 2019): 945–952. doi:10.1016/j.conbuildmat.2019.07.292.
[3] Fischel, M. "Evaluation of selected deicers based on a review of the literature." The Sea Crest Group, Colorado Department of Transportation, Denver, CO, Report No. CDOT-DTD-R-2001-15, 2001.
[4] L. Consultants and L. C. Limited, Guidelines for the selection of snow and ice control materials to mitigate environmental impacts vol. 577: Transportation Research Board, 2007.
[5] Shi, Xianming, David Veneziano, Ning Xie, and Jing Gong. “Use of Chloride-Based Ice Control Products for Sustainable Winter Maintenance: A Balanced Perspective.” Cold Regions Science and Technology 86 (February 2013): 104–112. doi:10.1016/j.coldregions.2012.11.001.
[6] Nilssen, Kine, Alex Klein-Paste, and Johan Wåhlin. “Accuracy of Ice Melting Capacity Tests: Review of Melting Data for Sodium Chloride.” Transportation Research Record: Journal of the Transportation Research Board 2551, no. 1 (January 2016): 1–9. doi:10.3141/2551-01.
[7] Muthumani, Anburaj, Laura Fay, Michelle Akin, Shaowei Wang, Jing Gong, and Xianming Shi. “Correlating Lab and Field Tests for Evaluation of Deicing and Anti-Icing Chemicals: A Review of Potential Approaches.” Cold Regions Science and Technology 97 (January 2014): 21–32. doi:10.1016/j.coldregions.2013.10.001.
[8] S. Institute, “Snowfighter’s Handbook,” A Practical Guide for Snow and Ice Control Alexandria, VA, (2012).
[9] Kelling D. L., Laxon C. L., Review of effects and costs of road de-icing with recommendations for winter road management in the Adirondack Park: Adirondack Watershed Institute. (2010).
[10] Ketcham S., Minsk L. D., Blackburn R. R., and Fleege E. J. "Manual of practice for an effective anti-icing program: a guide for highway winter maintenance personnel." United States. Federal Highway Administration (1996).
[11] Dumont M. "Canadian Minerals Yearbook (CMY)-2008: Salt." 2009.
[12] Blackburn, R. R., McGrane E. J., Chappelow C. C., Harwood D. W., and Fleege E. J.. "Development of anti-icing technology: Strategic Highway Research Program." National Research Council Washington, DC, (1994).
[13] Blackburn, R. R. "Snow and ice control: Guidelines for materials and methods vol. 526." Transportation Research Board, (2004).
[14] Hossain, S. M. K. "Optimum De-icing and Anti-icing for Snow and Ice Control of Parking Lots and Sidewalks. Waterloo." ON: PhD thesis, University of Waterloo, (2014).
[15] Gerbino-Bevins, B. M. "Performance Rating of De-icing Chemicals." M.Sc. Thesis, Department of Civil Engineering, The University of Nebraska, Lincoln, Nebraska, (2011).
[16] ASTM. "Standard Specification for Sodium Chloride." in D 632, ed. USA: American Society for Testing and Materials (ASTM), 2012.
[17] B. S. (BS). "Specification for salt for spreading on highways for winter maintenance." in 3274, ed. UK: British Standard, (2011).
Influence of Iron-Filings on Marshall and Volumetric Properties of Asphalt Concrete

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Abstract

The growth and expansion of road infrastructure had resulted in the continuous use of materials, increased construction costs of flexible pavements and increased environmental impact during the service life of the road. Consequently, many researchers have sought to use methods to maintain these roadways sustain environmental impact and traffic loads. One of these approaches is the use of additives to improve asphalt's volumetric character. In this research, iron filings were used as partial replacement of fine aggregates, and the Marshall and volumetric properties were assessed before and after the implementation of iron filings. Specimens were prepared with iron filings addition of (2, 4, 6 and 8 \%) by weight of fine aggregates. The Marshall mix design procedure was used to calculate the optimum asphalt content and the volumetric properties, including bulk density, Total voids, voids in mineral aggregates V.M.A., and voids filled with asphalt V.F.A. The Marshall Flow and Stability were calculated. Test results were assessed before and after the inclusion of the iron filings. It was concluded that the addition of iron filings can enhance the Marshall and volumetric properties of asphalt. The stability increased by 15\% when replacing fine aggregates by 2\%, of iron filings by total weight. Also, the air voids and the VMA decreased by increasing the percentage of iron filings, while VFA was not significantly affected as compared to the conventional specimen. The ideal ratio of iron filings which fulfill the optimal requirements was 5\%.

Keywords: Iron Filings; Volumetric Properties; Marshall; Stability; Flow; Mix Design.

1. Introduction

The advancement in material technology to support the sustainability in infrastructure design and construction has brought research work into focus. The major issue is to reserve the materials resources, reuse of reclaimed materials, reduce the need for energy, and improve the quality of the materials. One of the sustainable technologies is the use of iron scrap left in city-scattered iron-workshops. Disposal of iron filings is hazardous to the environment and proper disposal is difficult. Furthermore, iron filings can establish the self-repairing property for concrete due to their temperature conductivity. Many research studies have investigated the impact of incorporating iron waste on the quality of the flexible pavement. Jendia et al. (2016) \cite{1} studied the effect of adding steel wool SW to the asphalt concrete. Steel wool S.W. was added by (3.5 and 7\%) by weight of the asphalt to 20 samples. Volumetric, stability, and crawl characteristics of asphalt were calculated, as well as a study of its effect on the conductivity of asphalt and the extent of its effect on the self-healing property of asphalt. It was concluded that the rate of 5\% of S.W. had improved the conductivity of the asphalt.

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In comparison, the ratios 3 and 5% led to a stronger bond between the materials; As for the 4.3% of S.W., it was the ideal rate for improving the physical properties of asphalt. Alakhrass (2018) [2] investigated the influence of adding iron powder on asphalt concrete properties. The effect of repeated loads and fatigue was studied. (4, 8, 12 and 16%) of iron powder was added to the asphalt concrete specimens. Indirect tensile testing and permanent deformation were performed at (5, 25 and 40°C) temperatures and with the constant stress of 250 and 400 KPa. It was concluded that the performance of the modified asphalt fatigue is better than that of the traditional asphalt, as the life of fatigue increased by 100 and 150% for samples with a ratio of (8 and 12) of iron powder at a temperature of 5 °C when compared to conventional samples. Yan et al. (2019) [3] used steel slag in the asphalt concrete samples because of its benefits to the performance of the asphalt mixture, as well as to get rid of the costs of landfilling and environmental damage. Physical and mechanical tests were performed on samples with different content of the types of steel slag. A uniaxial compressive test was carried out, as well as an indirect tensile test to assess samples and study crack resistance at low temperatures and fatigue. It was concluded that the addition of furnace slag has increased the adhesion strength as well as improved shear resistance in modified samples compared to control samples. Afaf (2014) [4] added specific percentages of iron slag to asphalt concrete samples and examined the impact of this product on the stability and indirect tensile strength of asphalt samples. The rate of iron slag was (0, 10 and 20%) by the total weight of the mixture. It was concluded that there is an increase in stability when increasing the percentage of iron slag. A small increase was observed in the value of the indirect tensile strength when increasing the rate of iron slag.

Eisa (2018) [5] assessed the use of iron filings as a filler in the asphalt mixture of the wearing surface layer and studied its impact on the volumetric properties of the specimens. The filler weight for iron filings was (15, 20, 25, 35, 50 and 75%) of the mineral filler weight. It was reported that iron filings had changed the values of the volumetric properties of asphalt and the best percentage of iron filings that gave the best results was 25% by the weight of the filler. Arabani and Mirabolalzimi (2011) [6] used iron filings in the asphalt mixture at rates of (2.5, 5, 7.5, 10%) to replace the fine aggregates with this material and studied its impact on self-healing of micro-crack in the pavement. After preparing the specimens, they were cooled to (-20) °C and were fractured with a fracture test device, and the results were recorded. Samples were placed in the microwave for 90 seconds, and the temperatures were recorded. The specimens were fractured again with the fracture test device. The process was repeated, but the times in the microwave were changed to 120 and 140 seconds, and the results were compared. It was concluded that increasing the percentage of iron filings, which are conductive materials, has increased the temperature of the specimens at the fixed time. Also, fracture values were increased by increasing the content of iron filings.

Sarsam and Allamy (2020) [7] examined the effect of silica particles on pavement susceptibility to the fatigue cracking phenomenon. The results show that if the Silica fumes content was 1%, the fatigue life increases by 17%, and if the Silica fumes content increases to 2%, it increases by 46%. Therefore, the resilience of fatigue rises to 34% as Silica fumes rise to 3% relative to the control mixture. Remadevi et al. (2015) [8] studied the use of fibre reinforcement in asphalt concrete and its impact on stability, flow, and volumetric properties and compared the results with conventional asphalt concrete. 10mm polypropylene fibre was implemented at the rate of (4, 6, 8, 10%) by weight of bitumen. It was reported that using the optimum content of fibre of 5.33% by weight of bitumen and 4.41 % optimum asphalt content O.A.C. will increase the stability and decrease flow. Al-Tae et al. (2020) [9] studied the impact of two kinds of additives (Styrene-Butadiene-Rubber (S.B.R.) and carbon black) on the performance of recycled asphalt concrete mixture. It was concluded that the Resilient modulus (Mr) at (0.138 and 0.206) MPa stress level decreases by (14, 22 and 8) % and (22, 34 and 11) for reclaimed and recycle mixtures with (carbon black-asphalt and SBR-asphalt) respectively If compared to that at 0.068 MPa. In the asphalt mixture, Oluwasola et al. [10] added steel slag and copper mine residues by varying proportions and studied the effect of these additives on volumetric properties (air voids, specific gravity, V.M.A., V.F.A., Stability and Flow), Test results were evaluated and the f-test was used to assess the degree of importance of mixtures with both criteria. It was concluded that, the use of steel slag or copper mining residues or either has been shown to create better volumetric properties of asphalt mixtures. Xu et al. (2020) [11] used steel slag, steel fibre and basalt to replace fine and coarse aggregates in modified asphalt mixtures. The effect on mechanical, thermal and healing properties as well as the influence of moisture as a component of durability, and the natural cooling speed in the asphalt mixture was assessed. An infrared camera registered the heating and cooling velocities.

A Scanning Electron Microscope (S.E.M.) has detected the surface texture. Surface area and Porosimetry methods have measured the pore sizes. The percentage of steel fibre was 2%, 4% and 6% (by the volume of asphalt). It was concluded that Steel fibers have improved the moisture damage, high-temperature deformation tolerance and mechanical strength, while steel slag benefits from high-temperature deformation tolerance and mechanical performance, but is not moisture damage tolerant. Also, the healing process is successful in the induction of steel slag and asphalt concrete healing increases with decrease in steel fibres. Gao et al. (2018) [12] measured the heating efficiency of microwave asphalt mixtures comprising steel wool fibre. The effect of the additive was also evaluated in different ratios on the percentage of air voids . For the preparation of the asphalt mixtures, three types of low-carbon steel wool with five different diameters were used. It was concluded that the air spaces increase with increasing steel
wool ratios. Köfteci (2018) [13] used five asphalt samples with ratios (1, 3, 5, 7, 9%) of Low-Cost Iron Wire Fiber. The stability and moisture impact through an ITS and TSR were investigated. The findings of the investigations show an increase of 1%+3% increase in the quality of asphalt mixtures by incorporating low-cost iron fiber. When the rate of fibre increased by more than 3%, stereo-microscope clustering created by fibers was observed. As a result, air vacuums were increased and the interaction between bitumen and aggregates decreased. Especially increased fiber ratio caused compression, durability and stability problems in the mixture at 7-9 percent.

This work aims to assess the influence of partial replacement of fine aggregates with iron filings on the Marshall and volumetric properties of asphalt concrete. (2, 4, 6, 8%) of iron filings replaced fine aggregates of the same size. Seventy-five samples were prepared. The Marshall method of mixture design was used to calculate ideal asphalt content. Volumetric, stability, and flow characteristics of samples were assessed, and it was compared with the results of the conventional samples. Besides, the effect of different iron filings ratios on the behaviours of these properties was evaluated. Finally, it was suggested that the percentage of iron filings that achieve the optimum requirements for these properties will be evaluated. The research flow chart is shown in Figure 1.

![Research methodology flow chart](image)

Figure 1. Research methodology flow chart

2. Materials and Methods

2.1. Asphalt cement

Penetration grade asphalt cement (40-50) was obtained from the Al-Nasiriyah refinery. Details of physical properties of asphalt cement are included in Table 1.

| Property                      | Test Conditions          | ASTM, 2013 Designation | Test results | SCRB, 2003 Specification |
|-------------------------------|--------------------------|-------------------------|--------------|--------------------------|
| Penetration                   | 25°C, 100 gr, 5sec       | D5-06                   | 44           | 40-50                    |
| Softening Point               | -                        | D36-95                  | 49           | -                        |
| Ductility                     | 25°C, 5 cm/min           | D113-99                 | 140          | >100                     |
| Specific Gravity              | 25°C                     | D70                     | 1.03         | -                        |
| Flash Point                   | Cleave open land cup     | D92-05                  | 302          | >232                     |
| Retained Penetration of Residue (%) | 25°C, 100 gr, 5sec       | D5-06                   | 81           | >55                      |
| Ductility of Residue          | 25°C, 5 cm/min           | D113-99                 | 95           | >25                      |

Table 1. Physical properties of asphalt cement
2.2. Coarse and Fine Aggregates

The fine and coarse aggregates were taken from the stacks of the public Assyria Company of the Ministry of Construction and Housing. The required tests were performed and included in Tables 2 and 3.

| Property Value                                      | ASTM, 2013 Designation No. | Test results |
|----------------------------------------------------|-----------------------------|--------------|
| Bulk Specific Gravity of Coarse Aggregate          | C127-88                     | 2.619        |
| Apparent Specific Gravity of Coarse Aggregate      | C127-88                     | 2.687        |
| Absorption in per cent of Coarse Aggregate         | C127-88                     | 1 %          |
| % of Fractured Particles in Coarse Aggregate       | ASTM D5821-13               | 93%          |
| Resistance to Abrasion (Los Angeles)               | ASTM C131/C131M-2014        | 23%          |

Table 3. Properties of Fine Aggregates

| Property Value                                      | Test results |
|----------------------------------------------------|--------------|
| Bulk Specific Gravity of Fine Aggregate             | 2.621        |
| Apparent Specific Gravity of Fine Aggregate         | 2.694        |
| Absorption in per cent of Fine Aggregate            | 1.1%         |

2.3. Mineral Filler

Limestone dust has been used as filler and was collected from the Karbala factory. Basic testing for physical properties has been carried out. Table 4 illustrates its physical characteristics.

| Table 4. Properties of Mineral Filler |
|---------------------------------------|
| Per cent passing sieve No. 200        | 95           |
| Specific surface area (m²/Kg)         | 389          |
| Specific gravity                      | 2.850        |

2.4. Iron Filings

The iron filings used in this work were obtained from the local iron workshop in Baghdad. Sieve analysis was performed, and only the iron filings were used which passed sieve No. 8 and retained on the sieve number 50. Were implemented. Iron filings were used as a partial substitute for the fine aggregates of the same size as that of the iron filings. The density of the iron filings was 7.15 gr/cm³. Its calculation is based on CEN EN 1097-6/A1 [14]. Figure 2 illustrates the iron filings used in this work.

2.5. Selection of Combined Gradation

Asphalt concrete was prepared for wearing course type III-B according to the gradation limitations of SCRB [15] listed in Table 5. Figure 2 also shows the limitations used for the fine and coarse aggregate of the surface layer type III-B.

| Table 5. SCRB, 2003 Limitations of Aggregate Gradation |
|--------------------------------------------------------|
| Sieve size (mm)| Asphalt stabilized base course Type I | Binder course Type II | Wearing course Type III-A | Wearing course Type III-B |
|----------------|--------------------------------------|-----------------------|---------------------------|---------------------------|
| 37.5           | 100                                  | 100                   | 100                       | 100                       |
| 25.0           | 90-100                               | 100                   | 100                       | 100                       |
| 19.0           | 76-90                                | 90-100                | 100                       | 100                       |
| 12.5           | 56-80                                | 70-90                 | 90-100                    | 100                       |
| 9.5            | 48-74                                | 56-80                 | 76-90                     | 90-100                    |
| 4.75           | 29-59                                | 35-65                 | 44-74                     | 55-85                     |
| 2.36           | 19-45                                | 23-49                 | 28-58                     | 32-67                     |
| 0.300          | 5-17                                 | 5-19                  | 5-21                      | 7-23                      |
| 0.075          | 2-8                                  | 3-9                   | 4-10                      | 4-10                      |
2.6. Marshal Mix Design

The Marshall Mixture design method is a critical way to know the optimal asphalt content used for asphalt. The design method for Marshall Mixes comprises of six key elements:

1. Choosing the right fine and coarse aggregate
2. Successful selection of asphalt binder with laboratory tests
3. Preparing and compaction of the samples
4. A stability and flow test for samples
5. Calculations of Density and Air Voids
6. Optimum rate of asphalt selection

In this work, fine and coarse aggregates were oven dried at 110 °C and combined with specific weights and proportions. The sums have been combined to satisfy the defined wear rate according to SCRB, [15]. The combined aggregates and bitumen were heated to 150 °C; the required amount of asphalt was added to the sums and mixed for two minutes by hand over the hot plate. The iron filings were added at (0, 2, 4, 6, 8%) of the total weight of the mixture as a partial substitute of fine aggregates. All the materials were mixed at a temperature of 150 °C. Marshall moulds measuring (6.25 height and 10.16 diameters) were prepared, and these moulds were heated to the temperature of 140 °C on the hot plate. The mixture was placed in the moulds and stacked with a spatula, according to ASTM (2013) [16]. Non-absorbable papers were placed on the moulds. A Marshal hammer was used to compact the samples 75 blows on each face of the mould. The temperature limit was taken into consideration, not to fall below 135 °C.

The specimens were left to cool in the laboratory for 24 hours; then, the moulds were removed. In this study seventy-five samples were prepared with different contents of asphalt and iron filings to calculate the optimal content of asphalt in the presence of iron filings and determine the optimal iron filing percentage that meets the specifications of the Marshall Mix design. The volumetric properties of these samples were calculated as follows:

Bulk Specific Gravity ($G_{mb}$) was determined based on the weight of the samples in the air and their weight in the water from the following equation, as reported by Robert et al. (2009) [17].

$$G_{mb} = \frac{A}{B - C}$$  \hspace{1cm} (1)

Where:

$G_{mb}$= bulk specific gravity of the compacted mixture;
A= mass of the dry Specimen in the air (g);
B=mass of the saturated surface dry specimen in the air (g);
C=mass of the Specimen in water (g).

The theoretical maximum specific gravity of bituminous paving mixtures (Gmm) for each Specimen with different
content of asphalt and iron filing was calculated using the ASTM D2041 / D2041M-13 and the following equation:

\[ G_{mm} = \frac{A}{A-(C-B)} \]  

(2)

Where:

- \( G_{mm} \) = maximum specific gravity of the mixture;
- \( A \) = mass of dry sample in air, g;
- \( B \) = weight of bowl underwater, g;
- \( C \) = mass of container and sample underwater, g.

2.7. Air Voids

As stipulated in Al-Tae et al. (2020) [9], the air voids range from 3-5%. A.V value was extracted by changing the content of asphalt in the proportions of (4, 4.5, 5, 5.5, and 6) % of the samples. The content of iron filings in the previously mentioned proportions through the following equation:

\[ A.V = \left[ 1 - \frac{G_{mb}}{G_{mm}} \right] \times 100 \]  

(3)

Where:

- A.V= Percentage of air voids (%);
- \( G_{mm} \) = Maximum specific gravity of the hot asphalt mix ASTM D 2041 2011;
- \( G_{mb} \) = Bulk specific gravity of the compacted mixture.

2.8. Void in Mineral Aggregate (V.M.A.)

Lack of V.M.A. can influence the stability of the asphalt mixture [16, 17] was used to calculate voids in the mineral aggregate (V.M.A.) as follow:

\[ VMA = 100 - (G_{mb} \times \frac{P_s}{G_{sb}}) \]  

(4)

Where:

- \( G_{mb} \) = bulk specific gravity of the compacted mixture;
- \( G_{sb} \) = bulk specific gravity of the aggregate;
- \( P_s \) = percentage of the aggregate, by the total mass of mixture (%).

2.9. Void Filled with Asphalt (V.F.A.)

V.F.A. is associated with air spaces filled with asphalt. The V.F.A. is determined using the Equation (5) [16]:

\[ VFA = \frac{VMA-A.V}{VMA} \times 100 \]  

(5)

3. Results and Discussion

3.1. Unit Weight

Figure 3 shows the relationship between the unit weight and the asphalt content of samples with different proportions of iron filings. The value of the unit weight increases by increasing the percentage of asphalt until it reaches the highest value and then begins to descend. Figure 4 shows the relationship between the unit weights with the content of iron-filings in the asphalt. It can be observed that there is an increase in the unit of weight by increasing the content of iron filings. After reaching the highest value, it starts to descend. The reason for the rise in the unit weight is due to the high density of iron filings.

It can be observed that increasing the iron filing percentage will increase the density as long as the specimen can be compacted to a point where compaction is impossible due to protrusion ruggedness and thus density starts to decrease.6 per cent of the iron filings showed the highest unit weight value. Similar findings were reported by Jendia et al. (2016) [1].
Figure 3. Unit Weight versus Asphalt Content

Figure 4. Unit Weight versus Iron Filings

3.2. Air Voids

Figure 5 shows the relationship of the air voids with the asphalt content of samples with different proportions of iron filings. On the other hand, Figure 6 shows the relationship between the iron filings content and the air voids where there is a decrease in the percentage of air spaces with an increase in iron filings. The explanation for the reduction in air voids is the smoothness of iron filings. Low air avoids can lead to bleeding, rutting and loss of stability in the mix [12]. In comparison, high air vacuums can lead to durability and stripping problems. Also Brown et al. (2004) [18] stated that high air vacuums increase the penetration of air and water into the concrete. The required air vacuum quality must, therefore, be achieved during construction by applying compaction pressure. The results were not identical to Gao et al. (2019) and Jendia et al. (2016) findings [1, 12].
3.3. VMA & V.F.A.

Figures 7 and 8 show the relationship between asphalt content versus V.M.A. and V.F.A., respectively. Figures 9 and 10 show the relationship between the content of iron filings, V.M.A., and V.F.A., respectively. The reason for the lack of VMA when increasing the percentage of iron filings in the asphalt mixture is the increase in bulk specific gravity. The decrease in VMA leads to problems in durability, but the results were within specifications. The variation in VFA is not significant as compared to the traditional mixture sample, where the increase was from 72 to 74 when adding 8% iron filings to the asphalt mixture. The results obtained were different from Jendia et al. (2016) findings [1].

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**Figure 5. Air Voids versus Asphalt Content**

**Figure 6. Air Voids versus Iron Filings content**

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Figure 7. VMA versus Asphalt content

Figure 8. VFA versus Asphalt content

Figure 9. VMA versus Iron Filings Content
3.4. Stability and Flow

Among the most important outputs of the Marshall design of the mixture is the stability and flow of the samples. Figure 11 and Figure 12 illustrate the relationship between the content of asphalt with stability and flow, respectively. In contrast, Figure 13 and Figure 14 shows the relationship between the content of iron filings with stability and flow, respectively. It can be noted that the stability increases with an increase in the content of iron filings until it reaches the maximum value and then begins to descend. At the same time, the flow decreases with an increase in the content of iron filings. The highest stability was in samples with an iron filing content of 2%, while the lowest value of flow was for samples with an iron filing content of 4%. Table 6 shows the rate of increase in the stability of samples with different ratios of iron filings compared to the control sample (without adding iron filings). The improvement in stability with the addition of iron filings can be attributed to the rise in internal friction due to the angular form of the iron filings. Also, the reason for the decrease in stability in samples with the content of iron filings 6% and 8% is the property of non-absorption of binder in iron filings where if it exceeds the ideal percentage, the non-absorbed asphalt will increase and thus stability will decrease. In other words, the reduction in stability after the optimum iron fillings content can be clarified by the trapping of iron filings across the aggregate during mixing, contributing to a slight lack of connection between the blend. Similar results were reported by Jendia et al. (2016) [1].
Figure 12. Flow versus Asphalt content

Figure 13. Iron Filings versus Stability

Figure 14. Flow versus Iron Filings
Table 6. Impact of iron filings on Marshall Stability

| No. | Percent of Iron Filings | Marshal Stability (K.N.) | Percent Increasing |
|-----|-------------------------|--------------------------|-------------------|
| 1   | Control (0%)            | 10.7                     | -                 |
| 2   | 2%                      | 12.3                     | 15%               |
| 3   | 4%                      | 11.5                     | 7.5%              |
| 4   | 6%                      | 10.3                     | -3.74%            |
| 5   | 8%                      | 10                       | -6.54%            |

After calculating the average of the maximum values in the tables (unit weight, stability) with the content of asphalt, which achieves 4% of the air voids, the optimum content of the asphalt was obtained. For various concentrations of iron filings, Table 7 indicates the optimum asphalt standard.

Also, an ideal iron filing material that achieves optimum values for Marshall Mix requirements has been established. Table 8 shows the optimum iron filings content with volumetric properties, stability, and flow characteristics. The optimum content of iron filings was calculated by knowing the percentage of iron filings that achieve the highest stability and unit weight values, as well as the content that makes 4% air space and considering the average of the three values.

Table 7. O.A.C with different proportion of Iron Filings

| No. | Percent of Iron Filings | O.A.C  |
|-----|-------------------------|--------|
| 1   | 0%                      | 5.2%   |
| 2   | 2%                      | 5.1%   |
| 3   | 4%                      | 5%     |
| 4   | 6%                      | 4.9%   |
| 5   | 8%                      | 4.8%   |

Table 8. Volumetric Properties of Optimum asphalt & iron filings Specimen

| No. | Volumetric Properties | Value | Specification |
|-----|-----------------------|-------|---------------|
| 1   | % Iron Filings        | 5%    | -             |
| 2   | O.A.C                 | 4.95% | 4-6 %         |
| 3   | % Air Voids           | 4.1%  | 3-5 %         |
| 4   | Unit Weight           | 2.41  | -             |
| 5   | Marshal Stability     | 11.4  | 8 K.N. (min)  |
| 6   | Flow (mm)             | 2.8   | 2-4 mm        |
| 7   | VMA                   | 15.6  | 14% (min)     |
| 8   | V.F.A.                | 75    | -             |

4. Conclusion

The following conclusions may be may be drawn based on the limitations of materials and testing program. The use of iron filings led to a positive change in the properties of Marshall, as the stability and V.M.A. increased in comparison with the control sample. The value of bulk specific gravity increased, with an increase in the percentage of iron filings. The greatest value was when replacing fine aggregate with 6% iron filings. Also, it was found that there was no clear change in the V.F.A. ratio, with an increase in the iron filing rate as compared to the conventional sample. Air voids decreased with an increase in the percentage of iron filings when compared to the traditional sample, unlike previous studies that used steel wool as an additive. Stability increases with an increase in the percentage of added iron filings compared to the conventional sample, until it reaches its highest value and then begins to descend. The highest increase was seen in the samples with an iron filing content of 2%, with an increase of 15%. While stability decreased when using (6 and 8) % iron filings by (3.4 and 6.54) %, respectively. It was found that as iron filing content increases, The optimum content of Asphalt decreases. The optimum asphalt content of samples with iron filing content of (0, 2, 4, 6 and 8%) was (5.2, 5.1, 4.9, and 4.8%), respectively. The ideal percentage of iron filings that meet the Marshall Design mixture requirements as per SCRB was 5%, where the stability was 11.4 kN, the flow was 2.8 mm, V.M.A. was 15.6%, and V.F.A. was 75%. In addition to what has been mentioned, future studies should asses the effect of iron filings and other waste on the durability of asphalt during the service life.

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5. Conflicts of Interest

The authors declare no conflict of interest.

6. References

[1] Shafik Jendia, Noor Hassan, Khadija Ramlawi, and Hadeel Abu-Aisha. “Study of the Mechanical and Physical Properties of Self-healing Asphalt.” JOURNAL OF ENGINEERING 3, no. 4 (December 2016): 85-91.

[2] Alakhrass, M. S., “The Effect of Adding Iron Powder on Self-Healing Properties of Asphalt Mixture.”, Master’s Thesis, The Islamic University of Gaza-Faculty of Engineering-Civil Engineering Dept-Infrastructure Engineering. (2018).

[3] Yan, Zhou, and Zhang Hao. “Study on Preparation and Performance of Steel Slag Asphalt Mixture Based on Steel Slag Aggregate.” IOP Conference Series: Materials Science and Engineering 631 (November 7, 2019): 022067. doi:10.1088/1757-899x/631/2/022067.

[4] Afaf, A. H. M. “Studying the effect of steel slag powder on Marshall Stiffness and tensile strength of hot mix asphalt.” Journal of Engineering Sciences, Assiut University 42, no. 4 (2014): 575–581.

[5] Eisa, M. S., “Improving Asphalt Mix Properties Using Iron Filings A Mineral Filler.”” Journal of Computers and Structures, Assiut University 42, no. 4 (2014): 575–581.

[6] Arabani, M., and S.M. Mirabdolazimi. “Experimental Investigation of the Fatigue Behaviour of Asphalt Concrete Mixtures Containing Waste Iron Powder.” Materials Science and Engineering: A 528, no. 10–11 (April 2011): 3866–3870. doi:10.1016/j.msea.2011.01.099.

[7] Sarsam S. I. and Allamy A. K., “Fatigue Behavior of Modified Asphalt Concrete Pavement”. Journal of Engineering, Vol. 22(2) (2016): 1-10

[8] Remadevi M., Mathew, A., Arya, M.G, Babu, B., and Febymol K.B., “Determination of Optimum Bitumen Content of Fiber Reinforced Bituminous Concrete,” International Journal of Engineering Research and Development 11 No. 3 (March 2015): 73-82.

[9] Al-Tae, Mustafa Shakir Mahdi, and Saad Issa Sarsam. "Influence of Additives on Permanent Deformation and Resilient Modulus of Recycled Asphalt Concrete." Journal of Engineering 26, no. 2 (2020): 159-175.

[10] Oluwasola, Ebenezer Akin, Mohd Rosli Hainin, Md Maniruzzaman A. Aziz, and M. Naqiuddin M. Warid. "Volumetric properties and leaching effect of asphalt mixes with electric arc furnace steel slag and copper mine tailings." Sains Malaysiana 45, no. 2 (2016): 279-287.

[11] Xu, Haiqin, Shaopeng Wu, Hechuan Li, Yuechao Zhao, and Yang Lv. “Study on Recycling of Steel Slags Used as Coarse and Fine Aggregates in Induction Healing Asphalt Concretes.” Materials 13, no. 4 (February 17, 2020): 889. doi:10.3390/ma13040889.

[12] Gao, Jie, Haoyuan Guo, Xiaofeng Wang, Pei Wang, Yongfeng Wei, Zhenjun Wang, Yue Huang, and Bo Yang. “Microwave Deicing for Asphalt Mixture Containing Steel Wool Fibers.” Journal of Cleaner Production 206 (January 2019): 1110–1122. doi:10.1016/j.jclepro.2018.09.223.

[13] Köfteci, Sevil. “Experimental Study on the Low-Cost Iron Wire Fiber Reinforced Asphalt Concrete.” Teknik Dergi (July 1, 2018). doi:10.18400/tekderg.350135.

[14] CEN EN 1097-6/A1. “Tests for Mechanical and Physical Properties of Aggregates” – Part 6: Determination of particle density and water absorption. Brussels: C.E.N. (2005).

[15] SCRIB, “Standard Specification for Roads and Bridges.” Section R/9, Revised Edition. State Commission of Roads and Bridges, Ministry of Housing and Construction, Republic of Iraq (2003).

[16] ASTM, Road, and Paving Materials. Annual Book of ASTM Standards, (2013), Volume 04.03, American Society for Testing and Materials, U.S.A.

[17] Roberts, Freddy L., Prithvi S. Kandhal, E. Ray Brown, Dah-Yinn Lee, and Thomas W. Kennedy. “Hot mix asphalt materials, mixture design and construction.” 2nd Edition, NAPA Education Foundation, Lanham (2009).

[18] Brown, E. Ray, M. Rosli Hainin, Allen Cooley, and Graham Hurley. "Relationships of HMA in-place air voids, lift thickness, and permeability.” National Cooperative Highway Research Program (2004): 9-27.
Effect of Silica Fume on Permeability and Microstructure of High Strength Concrete

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Abstract

The important concrete structure in the vicinity of industry, thermal power plant suffers deterioration by the acid rain cause due to combination of CO₂, SOₓ and NOₓ with rain water. A combined attack that is from acid as well as sulphate can be observed under impact of sulphuric acid. It attacks on Calcium hydroxide and form Calcium sulphate, which can be leached out easily and make Interfacial Transition Zone (ITZ) poor. The water retaining structure such as dam, weir should be impermeable and that can be achieved by binary cementitious blends, using Silica fume (SF). Silica fume a by product of silicon industry, proves very effective in improving the microstructure of concrete due to their finer particle size, approximately 100 times finer than cement particles. The SEM image of binary blended high strength concrete (HSC) with Silica fume shows the condensed packing of cement hydration product and a dense microstructure as compare to control mix. The water permeability test result reveals that there is about 87 percent reduction in the coefficient of permeability achieved by inclusion of 10% Silica fume (SF) by weight of cement. Rapid chloride penetration test (RCPT) has been performed to investigate the ingress of chloride ions into the concrete. There was significant reduction in chloride ions penetration recorded due to SF inclusion.

Keywords: Microstructure; Calcium-Silicate-Hydrate (C-S-H); Permeability; Interfacial Transition Zone (ITZ); Ca(OH)₂; Control Mix.

1. Introduction

Silica fume is extremely fine pozzolanic material with high silica content and hence is effectively use in high performance concrete. In the late sixty’s and mid seventy silica fumes was simply emitted into the atmosphere. But due to the strict environmental law’s its utilisation in concrete and other industries started. Through the research it has been proved that silica fumes in concrete induces high strength and reduces permeability and because of this, the silica fumes becoming the world most versatile mineral admixture. Concrete is virtually a permeable material. Permeability of concrete usually refers to the degree at which water or other aggressive substance (sulphates, chlorides ions, etc.) penetrates it, that plays a major role in the long-term durability of concrete. If the voids are interconnected, concrete becomes pervious. Permeability refers to the amount of water penetrate through concrete when the water is under pressure, and also to the ability of concrete to resist diffusion of any substance. This plays a significant role in durability because it controls the rate of entry of moisture that may contain aggressive chemicals and the movement of water during heating or freezing. Therefore, higher the permeability lesser will be the durability [1]. The partial replacement of cement with Supplementary Cementitious Material (SCM) such as SF decreases the permeability by the formation of C-S-H gel in the concrete mix which further put resistance against corrosion [2, 3]. SF is considered...
to produce low permeability concrete because of its high pozzolanic character and extreme fineness [4]. Moreover, the incorporation of SF appeared to fill the spaces between cement grains, this further leads to a reduction in water permeability and penetration of chloride ions [5].

The pozzolanic property of SF helps in the formation of C-S-H gel and as a result, there is a significant reduction in permeability [6]. An important parameter for evaluating the durability performance and service life of an existing concrete structure is permeability [7, 8]. There has not been much reporting on the permeability of HSC. Permeability of concrete is adequately explained by mainly four transport processes in concrete which are: transport of water under a hydrostatic pressure head; transport of water by capillary suction; diffusion of ions under a concentration gradient; and transport of ions by moving fluid [9, 10]. It is well accepted that permeability is a good indicator of durability potential [11]. It is known that the permeability deteriorates durability of concrete in the aggressive environment. This is due to the fact that the transport processes in concrete govern the deterioration processes like carbonation, chloride attack and sulphate attack. Due to the application of fillers and pozzolanic materials to the concrete mix, strength and other properties of concrete get improved, as SF possesses both these property, due to which it improve the microstructure as well as make concrete impermeable [12]. Therefore, the blending of Portland cement with pozzolanic materials becomes an increasingly accepted practice in the construction of structures exposed to aggressive environments. There is drastic reduction in permeability of concrete when silica fume replacement level is equal to 8% by weight of cement. However, the permeability increases beyond this replacement level. Silica fume fineness is an important factor for permeability of silica fume concrete and further concluded that permeability decreases if silica fumes fineness increases [13]. The silica fume addition reduced the permeability of concrete by a very large value. The silica fume in concrete reduced the size of capillary pore and increases the probability of transforming continuous pores into discontinuous ones, as capillary porosity is related to permeability, thus permeability reduced [14].

Hence, in this study, effect of silica fume addition in concrete mix on microstructure of concrete have been investigated with the help of scanning electron microscope (SEM) image and try to investigate the mechanism behind reduction in pores and interconnected voids present in concrete matrix. The other objective of this study is to study the influence of SF inclusion on permeability of high strength concrete.

2. Experimental Program

2.1. Materials

The constituent material used for this study were Ordinary Portland Cement (OPC) 43 Grade, nano silica, fine aggregate of zone III, 20 mm graded coarse aggregate and super plasticizer Structuro 203 (polycarboxylic based). The OPC 43 conforming to [15] used in this study has specific gravity 3.15, fineness 0.225 m²/g and soundness (outoclave expansion) 0.8%. The Silica fume used in the experiment satisfied the requirement of Indian Standard [16] is purchased from Elkem South Asia Pvt. Ltd. The chemical composition of cement has been shown in Table 1 and the properties of SF have been shown in Table 2.

River sand from Sone river bed was used as fine aggregate and after sieve analysis confirms to zone III as per Indian standard. Locally available crushed stone “with maximum graded size of 20 mm have been used as coarse aggregate The sieve analysis conducted on coarse aggregate sample and confirmed to 20 mm graded size as per Indian standard [17]. The physical properties of fine and coarse aggregates have been reported in Tables 3 and 4 respectively.

| Table 1. Chemical composition of OPC |
|-------------------------------------|
| Chemical composition by mass (%)   |
| SiO₂  | K₂O+Na₂O | Al₂O₃ | SO₃  | CaO  | Fe₂O₃ | MgO  | Loss on ignition |
| 22.11 | 1.09     | 5.2   | 3.46 | 64.34| 3.45  | 2.61 | 1.45           |

| Table 2. Properties of Silica fume |
|-----------------------------------|
| Sl. No. | Parameters                  | Specification | Analysis |
| 1       | Silicon dioxide, SiO₂       | Min 85%       | 87.01%   |
| 2       | Moisture content            | Max 3%        | 0.57%    |
| 3       | Loss of ignition            | Max 6%        | 0.99%    |
| 4       | Physical requirement (>45mm) | Max 10%      | 1.13     |
| 5       | Pozzolanic activity index   | Min 105%      | 132      |
| 6       | Specific surface (m²/g)     | Min15m²/g     | 19.4     |
| 7       | Bulk density (Kg/m³)        | 500-700Kg/m³  | 616      |
Table 3. Physical property of Fine aggregate

|                | Specific gravity | Fineness modulus | Water absorption |
|----------------|------------------|------------------|------------------|
|                | 2.66             | 2.506            | 1.35%            |

Table 4. Physical property of Coarse aggregate

| Aggregate Crushing value | Aggregate Impact value | Specific gravity | Water absorption |
|--------------------------|------------------------|------------------|------------------|
|                          | 24%                    | 2.72             | 0.76%            |

2.2. Mix Proportion

Twelve mixes were prepared for M60 concrete by different permutation and combination of constituent materials and following IS code [18] with six replacement ratio of silica fume, viz. 0%, 2%, 4%, 6%, 8% and 10% by weight of cement and two level of w/b ratio, viz. 0.36 and 0.40. The Reduction in water-cement ratio to achieve higher strength is going to reduce the workability, but the advent of HRWR (High rate water reducer) allows it up to 0.36. The mix proportion for M60 concrete has been summarized in Table 5. Proper mixing of the concrete constituents is necessary for attaining the maximum strength.

Table 5. Different mix proportion for M60 concrete

| Mix Code | %SF | w/b ratio | C (kg) | SF (kg) | W (kg) | FA (kg) | CA (kg) | HRWR (kg) |
|----------|-----|-----------|--------|---------|--------|---------|---------|-----------|
| L-S0N0   | 0   | 0.36      | 377.78 | 0       | 135.68 | 782.24  | 1205.84 | 7.56      |
| L-S2N0   | 2   | 0.36      | 370.22 | 7.56    | 135.68 | 782.24  | 1205.84 | 7.56      |
| L-S4N0   | 4   | 0.36      | 362.67 | 15.11   | 135.68 | 782.24  | 1205.84 | 7.56      |
| L-S6N0   | 6   | 0.36      | 355.11 | 22.67   | 135.68 | 782.24  | 1205.84 | 7.56      |
| L-S8N0   | 8   | 0.36      | 347.56 | 30.22   | 135.68 | 782.24  | 1205.84 | 7.56      |
| L-S10N0  | 10  | 0.36      | 340    | 37.78   | 135.68 | 782.24  | 1205.84 | 7.56      |
| H-S0N0   | 0   | 0.40      | 349.8  | 0       | 139.92 | 801.87  | 1199.64 | 7         |
| H-S2N0   | 2   | 0.40      | 342.81 | 6.99    | 139.92 | 801.87  | 1199.64 | 7         |
| H-S4N0   | 4   | 0.40      | 335.81 | 13.99   | 139.92 | 801.87  | 1199.64 | 7         |
| H-S6N0   | 6   | 0.40      | 328.81 | 20.99   | 139.92 | 801.87  | 1199.64 | 7         |
| H-S8N0   | 8   | 0.40      | 321.82 | 27.98   | 139.92 | 801.87  | 1199.64 | 7         |
| H-S10N0  | 10  | 0.40      | 314.82 | 34.98   | 139.92 | 801.87  | 1199.64 | 7         |

2.3. Methodology

The guidelines of IS: 3085-1965 [19] has been followed in this study, to measure the permeability of concrete cube of different mix proportion in the laboratory. In the laboratory to measure the coefficient of permeability of test specimen, ‘concrete permeability test apparatus’ (AIM-381) manufactured by Aimil Ltd. has been used. For water permeability test, cubical concrete specimen of size 150mm were caste in the mould and coefficient of permeability has been find at 28 days of water curing. Chloride ion penetration was measured at various ages using the rapid chloride permeability test in accordance with AASHTO T 277 (recently adopted as ASTM C 1202-91). This test does not offer diffusion constant, but rather an index, which has been found useful in comparative studies [20]. For RCPT cylindrical specimen of diameter 100mm and height 50 mm have been casted in lab and test were done at 56 days of water curing. The microstructural studies have been done with the help of SEM photograph of control concrete and silica fume concrete at 7 days and 28 days.

3. Results and Discussion

The results of water permeability test and rapid chloride penetration test obtained from this study have been summarized in Table 6.
Table 6. Permeability test result for all mix code

| Mix Code | % SF | w/b ratio | RCPT (coulomb) | Coefficient Of permeability (cm/s) |
|----------|------|-----------|---------------|----------------------------------|
| H-S0N0   | 0.0  | 0.40      | 2425          | 4.87E-09                         |
| H-S2N0   | 2.0  | 0.40      | 2232          | 4.25E-09                         |
| H-S4N0   | 4.0  | 0.40      | 2080          | 1.288E-09                        |
| H-S6N0   | 6.0  | 0.40      | 1752          | 1.127E-09                        |
| H-S8N0   | 8.0  | 0.40      | 1360          | 8.35E-10                         |
| H-S10N0  | 10.0 | 0.40      | 1138          | 6.15E-10                         |
| L-S0N0   | 0.0  | 0.36      | 2390          | 4.52E-09                         |
| L-S2N0   | 2.0  | 0.36      | 2132          | 4.12E-09                         |
| L-S4N0   | 4.0  | 0.36      | 1986          | 1.195E-09                        |
| L-S6N0   | 6.0  | 0.36      | 1692          | 1.072E-09                        |
| L-S8N0   | 8.0  | 0.36      | 1285          | 7.36E-10                         |
| L-S10N0  | 10.0 | 0.36      | 1088          | 5.18E-10                         |

3.1. Water Permeability Test Result

The variation in coefficient of permeability of cube specimens with varying percentage of silica fume and w/b ratio has been shown graphically in Figure 1. The variation in coefficient of permeability shows that as percentage of silica fume increases permeability reduces and for same percentage of SF, as w/b ratio reduces the permeability also reduces. From the result obtained it is clear that the coefficient of permeability is maximum for the control mix compared to all other mixes and as the percentage of silica fume increases, the permeability reduces. Concrete with silica fume shows good resistance against water permeation, it can be attributed to the high specific surface area of silica fume which results in greater pozzolanic activity and due to its finer size it fills the void between cement particle.

3.2. Rapid Chloride Penetration Test Result

The variation in charge passed through specimen in six hours with varying percentage of silica fume and w/b ratio has been shown graphically in Figure 2. From the graph it is infer that as percentage of silica fume increases, the ingress of chloride ions reduces and for same percentage of SF, as w/b ratio increases the charge passed increases. From the result obtained it is clear that the charge passed is maximum for the control mix compared to all other mixes and as the percentage of silica fume increases, the ion penetration reduces.

Figure 1. Water permeability test result for different mixes
3.3. Scanning Electron Microscope (SEM) Test Result

After the permeability test result, it has been prove that the concrete shows best resistant against water and ion permeation at 8% replacement level of silica fume and w/b ratio 0.36. So in this study the SEM study has been done for Mix Code L-S8N0. To understand the changing occurs in the microstructure of concrete matrix by the application of SF, we firstly investigate the microstructure of Control Mix at 7 days and 28 days of curing and SEM photograph of control mix have been shown in Figure 3. The SEM photographs of SF concrete have been shown in Figure 4.

![Figure 3. SEM photograph of Control concrete](image1)

a) Microstructure of Control mix at curing age of 7 days  
b) Microstructure of Control mix at curing age of 28 days

![Figure 4. SEM photograph of Silica fume concrete](image2)

a) Microstructure of Mix L-S8N0 at curing age of 7 days  
b) Microstructure of Mix L-S8N0 at curing age of 28 days
From the SEM image of control mix (L-S0N0) it can be seen that there is a heterogeneous distribution of C-S-H, CH grains and needle like ettringite crystals. There are some micro cracks are visible within structure. From the SEM image of control mix L-S0N0 at 28 days of curing, it can be seen that the deposit of small and large size CH crystals are dispersed in the hardened state. It is pertinent to mentioned that, there is an insignificant growth of C-S-H gel recorded from 7 days to 28 days of curing.

The SEM image of SF concrete at 7 days of curing, shows the condensed packing of cement hydration products. The mineral particle of SF is arbitrarily dispersed throughout the hydrated cement products. The Figure 4(a) shows the CH and Etrtringite needle formations crystals found between the C-S-H crystals, which contribute in the strength development. From the Figure 4(b), it is evident that although the incomplete reaction of SF due to controlled hydration, the formation of Ca(OH)$_2$ is less in comparison with Portland cement hydration and efficiency of its utilisation is improved with the presence of silica oxide. So a dense micro structure is evident in the SEM image presented in Fig 4(b), due to the filler effect of SF also. So increment in durability observed through the water permeability test and RCPT tacitly conform through the images.

4. Conclusion

On the basis of the experimental results, it can be infer that as the percentage of silica fume increases in concrete mix, the water and ion permeability reduces, but the rate of reduction in permeability reduces after 8% silica fume inclusion. So the 8% SF replacement level proves optimum dose for best resistance against water and ion permeation, this finding is in line with the observation of Song et al. (2010) [13]. It can also be concluded that as the w/b ratio reduces, the permeability also reduces. But when we reduces w/b ratio beyond 0.36, the concrete became unworkable.

From the microstructural study, it may be attributed to the fact that SF not only improves the pore structure in cement mortar but also improves the cement mortar/aggregate interface. Further, SF makes the cement mortar and transition zone dense and homogeneous due to its micro filler and pozzolanic effect by arresting the calcium hydroxide produced during hydration process of Portland cement and formation of C-S-H gel, this observation is in accordance with the findings of Khayat et al. (1997) [6], thereby making concrete almost impermeable.

5. Conflicts of Interest

The authors declare no conflict of interest.

6. References

[1] Miloud, B. "Permeability and porosity characteristics of steel fiber reinforced concrete." (2005): 317-330.
[2] Malhotra, V. M., and P. K. Mehta. "Advances in Concrete Technology, vol. 1." Pozzolanic and Cementitious Materials, 1st edn. Gordon and Breach Science Publishers, New York (1996).
[3] Kirkbridge, T. W. "Condensed silica fume in concrete: FIP state of art report." London: FIP commission, UK: Thomas Telford House (1988).
[4] Hou, Jiangyuan, and D.D.L. Chung. "Effect of Admixtures in Concrete on the Corrosion Resistance of Steel Reinforced Concrete." Corrosion Science 42, no. 9 (September 2000): 1489–1507. doi:10.1016/s0010-938x(99)00134-1.
[5] de Gutierrez, R. Mejia. "Effect of supplementary cementing materials on the concrete corrosion control." Revista de metalurgia 39 (2003): 250-255.
[6] Khayat, K. H., M. Vachon, and M. C. Lanctot. "Use of blended silica fume cement in commercial concrete mixtures." ACI Materials Journal 94 (1997): 183-192.
[7] Chia, Kok Seng, and Min-Hong Zhang. “Water Permeability and Chloride Penetrability of High-Strength Lightweight Aggregate Concrete.” Cement and Concrete Research 32, no. 4 (April 2002): 639–645. doi:10.1016/s0008-8846(01)00738-4.
[8] Hearn, Nataliya, Rachel J. Detwiler, and Carmen Sframeli. “Water Permeability and Microstructure of Three Old Concretes.” Cement and Concrete Research 24, no. 4 (1994): 633–640. doi:10.1016/s0008-8846(94)90187-2.
[9] Halamickova, Pavla, Rachel J. Detwiler, Dale P. Bentz, and Edward J. Garboczi. “Water Permeability and Chloride Ion Diffusion in Portland Cement Mortars: Relationship to Sand Content and Critical Pore Diameter.” Cement and Concrete Research 25, no. 4 (May 1995): 790–802. doi:10.1016/0008-8846(95)00069-a.
[10] Salvoldi, B.G., H. Beushausen, and M.G. Alexander. “Oxygen Permeability of Concrete and Its Relation to Carbonation.” Construction and Building Materials 85 (June 2015): 30–37. doi:10.1016/j.conbuildmat.2015.02.019.
[11] Bentz, Dale P., Mark A. Ehlen, Chiara F. Ferraris, and Edward J. Garboczi. "Sorptivity-based service life predictions for concrete pavements." In Proceedings of the 7th International Conference on Concrete pavements, Florida, USA, vol. 1, (2001): 181-193.
[12] Aitcin, P-C., S. L. Sarkar, and P. Laplante. "Long-term characteristics of a very high strength concrete." Concrete International 12, no. 1 (1990): 40-44.

[13] Song, Ha-Won, Seung-Woo Pack, Sang-Hyeok Nam, Jong-Chul Jang, and Velu Saraswathy. "Estimation of the permeability of silica fume cement concrete." Construction and Building Materials 24, no. 3 (2010): 315-321. doi:10.1016/j.conbuildmat.2009.08.033.

[14] Paillere, A-M, M. Buil, and J. J. Serrano. "Effect of fiber addition on the autogenous shrinkage of silica fume." Materials Journal 86, no. 2 (1989): 139-144.

[15] Tech. Rep. IS: 8112 "Grade Ordinary Portland Cement-specification," Bureau of Indian Standards New Delhi, India, (2005).

[16] IS 15388, Specifications for Silica Fume, Bureau of Indian Standards (2003).

[17] IS 383, “Specification for Coarse and fine aggregates from natural sources for concrete,” Bureau of Indian Standards, (2016).

[18] IS 10262, Indian standards recommended Guidelines for concrete mix design, 2009th ed. Bureau of Indian Standards, (2009).

[19] IS:3085-196, “Method of Test for Permeability of Cement Mortar and Concrete,” Bureau of Indian Standards, (1965).

[20] D. Whitting, “Rapid Determination of the Chloride Permeability of Concrete,” in FHWA Report FHWA/RD-81/119, Federal Highway Administration, Washington D C, USA, (1981).
Study the Effect of Substitution Filler on performance of Asphalt Mixture

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Abstract

The major distresses in asphalt pavements are rutting, fatigue, and adhesion loss (moisture susceptibility). In this research study, two substitution fillers (Cement and Lime) were used with two different aggregate quarries (based on minerals composition) to evaluate the relatively most beneficial combination of both fillers as well as an aggregate quarry to enhance the performance life of asphalt pavements, especially in under-developed countries. Four basic tests, (Asphalt Pavement Analyzer, Four Points Bending Beam, Dynamic Modulus, and Rolling Bottle Test) that used for the most desired properties of any asphalt pavement, were utilized to access the performance properties of modified asphalt mixture. Based on all laboratory test results this research study concludes that replacement of aggregate filler with hydrated lime and cement has a beneficial effect on asphalt mix performance and to save investment by using raw material. Substitution filler improves the high-temperature rut performance and intermediate temperature fatigue performance of asphaltic concrete mixture up to 25% to that of the conventional mixture. At the same time, substitution filler has more beneficial to improve 70% adhesion properties to that of the conventional mixture.

Keywords: Substitution Filler; Rut Performance; Fatigue Performance; Adhesion Properties; Asphalt Mixture.

1. Introduction

Materials generally employed for the construction of road or airport surfaces are asphalt concrete, which is a composite form of material made from asphalt binder, aggregate, and mineral fillers. The word mineral filler is usually referred to as fine material passing sieve number 200 standard sieve sizes. This fine material contains more than 90% of the aggregate surface area and it forms most interfaces between aggregate and asphalt binder. It is also an essential material that provides better packing characteristics between the coarse and fine aggregate. To this point, researchers always try to find substitute filler for better asphalt mixture and to save investment by using the raw materials as filler. The filler is classified as fine material and used for the modification of bituminous binder and asphalt concrete mixture. So far, different types of materials such as clay particle and limestone dust are considered as filler material. Similarly, hydrated lime, fly ash, and cement are used as modified fillers. The use of ordinary Portland cement in asphalt mixture as a substitution filler can significantly increase the resistance to rutting [1]. The addition of different fillers such as hydrated lime, fly ash, silica fume, cement, and bag-house fines is known to increase the rut resistance.

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of asphalt mixture [2]. Characteristics of mineral filler and their impact on characteristics of bituminous mixture varies with amount and type of filler added to the mix [3]. Cement and hydrated lime are typically used for filler modification. The modifications of filler material applying in the asphaltic concrete mixture have been studied by some researchers. Lesueur et al. (2012) [4] reported that the durability of asphalt mixture can be increased up to 10 years when 1-2% of hydrated lime is used in asphalt concrete mixture. Similarly, the addition of cement into an emulsified asphalt mixture improves the high-temperature stability of the mixes [5]. It is also observed that the utilization of cement as some part of filler in asphalt mixture can improve the adhesion properties of asphalt mixture [6, 7].

Wang et al. (2018) [8] concluded in his research that the use of cement with filler material in asphalt mixture can improve stiffness, stripping resistance, and strength of asphalt concrete mixture. Moisture susceptibility and high-temperature stability of asphalt mixture significantly improved by the addition of cement [9]. Zhou et al. (2017) [10] analyzes and compared the physical characteristics of recycled fillers with limestone filler and found a large difference between these fillers and rheological properties of asphalt mixture. Some researcher uses a wet method of mixing for modification of asphalt binder by hydrated lime. Huang et al. (2005) and Kakade et al. (2016) [11, 12] presented that moisture resistance ability of bituminous mixture found much better when hydrated lime is directly added to asphalt binder. For a thick layer of pavements, mixing of hydrated lime by a wet method of mixing give better results as compared to the dry method of mixing for fatigue life. However, for thin layers, mixing 1-2% of hydrated lime as filler material gives better results of fatigue life [13]. The addition of 1-2% of hydrated lime in un-aged asphalt mixture can be beneficial for fatigue life improvement [14]. Wang et al. (2018) [8] concluded in his research study that cement filler has a similar effect to that of mineral filler on volumetric and physical properties of asphalt mixture. Hydrated lime has been widely used for the improvement of resistance against moisture damage in asphalt binder but very limited work has been presented on the use of hydrated lime for modification of asphalt mixture [15-17]. The permanent deformation performance of asphalt mixture can be improved by adding hydrated lime as filler into asphalt mixture [18-25]. Roy et al. (2013) [26] reported that the use of limestone dust as filler is very beneficial for water sensitivity of stone mastic asphalt mixtures and Vansteenkiste et al. (2013) [27] concluded that the use of hydrated lime as the filler has a greater beneficial effect on permanent deformation. The significant factors which separate the fillers from other materials are particle size, shape, surface area, void content, mineral, physical and chemical properties [28]. For this reason, various types of fillers are introduced into asphalt mixtures for performance improvement. The complete structure of this research article presented in Figure 1.
In this research study, two substitution fillers (Cement and Lime) were used with two different aggregate quarries (based on minerals composition) to evaluate the relatively most beneficial combination of both fillers as well as an aggregate quarry to enhance the performance life of asphalt pavements, especially in under-developed countries. Moreover, lime being a raw material will not only reduce the cost but will also be eco-friendly to society. Four basic tests (Asphalt Pavement Analyzer, Four Points Bending Beam, Dynamic Modulus and, Rolling Bottle Test) that test the most desired properties of any asphalt pavement i.e. Rut resistance, Fatigue crack resistance, and moisture susceptibility were utilized to access the performance properties of modified asphalt mixture.

2. Materials and Method

The material and method which is used for the preparation of laboratory specimens and testing procedures are described in the following subsection. The summary of the research methodology has been given in Figure 2.

![Figure 2. Research Methodology](image)

2.1. Materials

Two types of aggregate have been selected in this study as per ASTM D3515. The aggregate has been taken from two different quarries i.e. Margallah and Sargodha. The selected aggregate has different petrography/mineralogy. Magellan aggregate has calcium carbonate minerals and Sargodha aggregate has dolerite minerals. Petrography of selected aggregate is given in Table 1.

| Sr. No. | Component          | Formula    | Calcium Carbonate | Dolerite |
|---------|--------------------|------------|-------------------|----------|
| 1       | Carbonate & Calcite| CaCO₃ & CaO | 97.2              | 93.4     |
| 2       | Quartz             | SiO₂       | 0.3               | 6.6      |
| 3       | Hematite           | Fe₂O₃      | 0.5               | --       |
| 4       | Clay               | --         | 2                 | --       |

Table 1. Petrography of aggregate
These types of aggregate have been frequently used in Pakistan for rehabilitation and new road construction projects. Mechanical and physical properties of selected aggregates were determined as per the specification of BS, ASTM, and AASHTO standards. Properties of selected aggregate are summarized in Table 2. River bed and rounded particles were not selected for this research study.

Table 2. Properties of selected aggregate

| Sr. No. | Test Title                      | Standard              | Aggregate Sources | Unit | Specification Limit |
|---------|--------------------------------|-----------------------|-------------------|------|---------------------|
| 1       | Flakiness Index                 | BS 812.108            | Calcium Carbonate | 5.9  | 8.4                 | % | 10 (max)            |
| 2       | Los Angeles abrasion Value      | ASTM C 131            |                  | 24   | 23                  | % | 30 (max)            |
| 3       | Fractured Particles             | ASTM D 5821           |                  | 100  | 100                 | % | 90 (min)            |
| 4       | Sand Equivalent Value           | ASTM D 2419           | Dolerite          | 78   | 75                  | % | 50 (min)            |
| 5       | Water absorption                | ASTM C 127            |                  | 1.01 | 0.97                | % | 2 (max)             |
| 6       | Elongation Index                | BS 812.109            |                  | 2.2  | 6.4                 | % | 10 (max)            |
| 7       | Soundness (Coarse)              | ASTM C 88             |                  | 7.5  | 4.7                 | % | 8 (max)             |
| 8       | Soundness (Fine)                | ASTM C 88             |                  | 4.9  | 4.7                 | % | 8 (max)             |

All the mixtures were designed with the same aggregate gradation. The graphical and tabular description of aggregate gradation is given in Figure 3 and Table 3 respectively.

![Figure 3. Selected Aggregate Gradation](image-url)

Table 3. Adopted aggregate gradation

| Sieve Size | Selected Gradation passing (%) | Class B Lower passing (%) | Class B Upper passing (%) |
|------------|--------------------------------|---------------------------|---------------------------|
| Inch/mm    | (passing (%))                  | (lower passing (%))       | (upper passing (%))       |
| 3/4        | 19                             | 100                       | 100                       |
| 1/2        | 12.5                           | 77.4                      | 90                        | 75      |
| 3/8        | 9.5                            | 64.1                      | 80                        | 60       |
| #4         | 4.75                           | 39.9                      | 60                        | 40       |
| #8         | 2.36                           | 27.7                      | 40                        | 20       |
| #16        | 1.18                           | 16.6                      | 27                        | 12       |
| #30        | 0.6                            | 10.4                      | 19                        | 8        |
| #50        | 0.3                            | 8.1                       | 15                        | 5        |
| #100       | 0.15                           | 5.7                       | 11                        | 4        |
| #200       | 0.075                          | 4.1                       | 8                         | 3        |
Asphalt mixtures were designed with a 12.5mm nominal aggregate size. Six types of asphalt mixtures including three Margallah aggregate mixtures (one mixture having Margallah aggregate dust as filler, other having Portland cement as a filler, and third one having hydrated lime as filler material) and three Sargodha aggregate mixtures (one mixture having Sargodha aggregate dust as filler, other having Portland cement as a filler and third one having hydrated lime as filler material) were produced. The material smaller than sieve no. 200 was replaced with the same weight of cement and hydrated lime in mixtures having cement and hydrated lime as filler material respectively. The mixtures with Portland cement and hydrated lime as substitution fillers have been used to evaluate the effect of substitution filler on the performance of asphalt mixture. Similarly, an asphalt binder having performance grade 58-22 with penetration grade 60/70 were used in the study. The basic properties of bitumen have been reported in Table 4.

Table 4. Properties of control binder

| Sr. No. | Test Title                          | Standard       | Bitumen type (Pen. 60/70) | Specification Limit |
|---------|-------------------------------------|----------------|----------------------------|---------------------|
| 1       | Flash and fire point (°C)           | ASTM C 142     | 291                        | 232 (min)           |
| 2       | Softening Point (°C)                | ASTM D 36      | 46                         | ---                 |
| 3       | Ductility (cm)                      | ASTM C 88      | 100                        | 100 (min)           |
| 4       | Penetration at 25 °C (1/10th of mm)| ASTM D 5       | 70                         | ---                 |

For designing six asphalt mixtures, optimum bitumen content of 4.3% for Margallah aggregate and 4.8% for Sargodha aggregate minerals were determined from the Marshall mix design method [29]. A very little effect of filler substitution was found on asphalt content. In this study, the optimum bitumen content of 4.3% for Margallah aggregate and 4.8% for Sargodha aggregate minerals was fixed for all mixtures. Portland cement and hydrated lime were taken from locally available sources. Some basic properties of cement and lime are given in Table 5.

Table 5. Properties of Substitution Fillers

| Specific Property Name       | Standard     | Portland Cement | Hydrated Lime |
|------------------------------|--------------|-----------------|---------------|
| Density (lb/cu in)           | ASTM D792    | 0.1139          | 0.084-0.973   |
| Specific Heat (JK\(^{1}\)Kg\(^{-1}\)) | ASTM E1269   | 1554            | 913           |
| Water Absorption (%)         | ASTM D570    | 0.05-0.08       | 0.975         |
| Specific gravity             | ASTM D891    | 3.16            | 2.4-2.7       |
| Tensile strength (psi)       | ASTM D638    | 300-700         | 138.78        |
| Thermal Conductivity (W m\(^{-1}\)K\(^{-1}\)) | ASTM E1952   | 1.17            | 1.26-1.4      |
| Melting Point (°C)           | ASTM D1519   | 1450            | 2574          |

2.2. Sample Preparation

For four performance tests (APA, FPBB, DM, and RBT), samples were prepared according to their standard procedures. 7 kg aggregate samples were compacted for asphalt pavement analyzer and dynamic modulus by considering the procedure postulated in AASHTO PP 35.
Similarly, roller compacted slabs were prepared for the FPBB test. The limits of air voids in compacted samples were kept at 6±0.5%. The temperature of the mixture for mixing and compaction purposes was kept at 160±3°C and 150±2°C respectively. For the rolling bottle test, the particles passing sieve no.9.5 and retained on sieve no. 6.3 were selected. The description and dimensions of used samples are given in Table 6.

Table 6. Description and sample sizes

| Test Description                     | Standard      | Sample dimension/Size          | Air voids (%) |
|--------------------------------------|---------------|--------------------------------|---------------|
| Asphalt Pavement Analyzer (APA)     | AASHTO TP 63  | • Diameter 150 mm               | 6.03          |
|                                      |               | • Height 75 mm                  |               |
| Dynamic Modulus Test (DM)            | AASHTO TP 62  | • Diameter 100 mm               | 6.1           |
|                                      |               | • Height 150 mm                 |               |
| Four Point Bending Beam (FPBB)       | AASHTO T 321  | • (380x63x50) mm                | 6.04          |
| Rolling Bottle Test (RBT)            | BS EN 12697   | Particle passing sieve no.9.5 and retained on sieve no.6.3 | --- |

Some volumetric properties of asphalt mixture with treated and untreated fillers are summarized in Table 7.

Table 7. Hot mix asphalt design volumetric properties

| Mix Type     | Aggregate type       | OBC | VA  | VMA | VFA |
|--------------|----------------------|-----|-----|-----|-----|
| Standard     | ASTM D3515           | --- | 04 to 07 | ASTM D6995 | --- |
| Specification Limit | --- | --- | 04 to 07 | 14 (Minimum) | 65-75 |
| Conventional Mix | Calcium carbonate | 4.3 | 6.1 | 15.53 | 60.95 |
|               | Dolerite             | 4.8 | 6.0 | 15.85 | 62.30 |
| Lime as Filler | Calcium carbonate   | 4.3 | 6.1 | 16.61 | 63.29 |
|               | Dolerite             | 4.8 | 6.0 | 17.32 | 65.57 |
| Cement as Filler | Calcium carbonate | 4.3 | 6.1 | 16.54 | 62.92 |
|               | Dolerite             | 4.8 | 5.7 | 17.42 | 67.50 |

Super-pave gyratory compacted samples were cut into two pieces and get the required standard height for asphalt pavement analyzer. Similarly, roller compacted slabs were fabricated into the required dimensions.

3. Results and Discussion

3.1. Rut Performance of Asphalt Mixture

Rutting performance of asphalt mixtures was evaluated with the asphalt pavement analyzer. Different computer-operated equipment with temperature control chambers are used in the laboratory for performance measurement of asphalt mixtures at the approximately same condition to that in-service pavement. APA is multifunctional testing equipment used to predict the performance of hot mix asphalt (HMA) and simulate it with field performance. Super-pave gyratory compacted samples with 150mm diameter and 75mm height were loaded with a steel wheel of 100 lb. which resting on a pneumatic hose with 100 psi pressure. Rutting test was conducted at 55°C in accordance with AASHTO TP 63. The obtained results are given in Figure 5.
Figure 5. Asphalt Pavement Analyzer test results

Figure 5 illustrates the evaluation of rut depth against a number of wheel passes according to standard laboratory test conditions and clearly mentioned the improvement of rut resistance of mixture with substitute fillers as compared to mixtures with untreated fillers. The figure shows that rut depth increases with an increase in a number of wheel passes for all mixes. The Sargodha aggregate, containing dolerite minerals, without substitution filler have maximum rut depth against 8000 wheel passes as compared to Margallah aggregate containing Calcium carbonate minerals. All at once, when the cement and hydrated lime fillers were introduced into asphalt mixture, the rut depth values considerably decreases. It also has been noted from figure that when hydrated lime was used as filler material, dolerite minerals have less rut depth as compared to Calcium carbonate minerals. Similarly, cement and hydrated lime as substitution filler have a considerable effect on rut depth and improve the resistance against permanent deformation for all types of mixtures.

3.2. Dynamic Modulus

Two main components are typically described such as phase angle (δ) which gives elastic and viscous behavior of bituminous mixture and dynamic modulus which give material stiffness under different loading frequency and temperature conditions. The dynamic modulus is typically defined as “the ratio of the amplitude of sinusoidal stress at any given loading frequency and time to the sinusoidal strain at given the same frequency and time condition. Dynamic modulus [E*] is one of the important property of bituminous mixture for prediction of pavement performance under different temperature and frequency conditions. The behavior of temperature-time dependency of hot mix asphalt (HMA) is usually defined in the laboratory determine dynamic modulus. Due to changing frequency and temperature dependency, it is very difficult to compare results across varying frequency and temperature conditions. For this, a master curve is developed for meaningful comparison. Master curves is a means to obtain a visual representation of results taken from different frequency and temperature conditions [30]. For testing prepared specimens, four temperatures (4.4, 21.1, 37.8 and 54.4) and six frequencies (25Hz to 0.1Hz) were used. The test was performed in accordance with AASHTO TP 62. The master curve results are given in Figure 6.
Figure 6 shows the evaluation of dynamic modulus against reduced frequency at standard test condition and clearly observed that dynamic modulus increased with the introduction of cement and hydrated lime as substitution filler. A calcium carbonate mineral with hydrated lime as filler shows maximum dynamic modulus at both, high temperature, low-frequency condition and low temperature, high-frequency condition. The utilization of hydrated lime as filler in asphalt mixture has more significant beneficial effect on dynamic modulus as compared to cement filler.

3.3. Fatigue Performance of Mixture

Four-point bending beam (FPBB) test is used for the measurement of fatigue damage or stiffness loss of hot mix asphalt material during repeated loading. Fatigue is considered an important parameter during designing the purpose of asphalt pavement. The test was performed in stress control mode in accordance with AASHTO T 321. The temperature of the test chamber was set at 20±0.5°C. The beam was considered as a failure when 380mm long and 63mm wide by 50mm thick HMA specimen reduce its 50% initial stiffness. The failure energy and fatigue life of hot mix asphalt were determined under repeated load conditions. The test results are given in Figure 7.

![Fatigue life of the mixes](image)

It can be observed from Figure 7. Calcium carbonate minerals with hydrated lime as filler material show maximum resistance against fatigue cracking at standard test conditions. Both cement and hydrated lime as substitution filler improves the fatigue performance as compared to conventional asphalt mixture. Like the previous results, hydrated lime as a substitute filler gives better performance as compared to cement.

3.4. Adhesion Performance of Mixture

Adhesion properties of the mixes were evaluated by using a rolling bottle test (RBT). RBT is used to measure the aggregate-bitumen coating vulnerability after mechanical abrasion under-water. Rolling bottle test is a visual inspection test from which the remaining covering of aggregate-bitumen coating is estimated. The test was performed in accordance with European standard BS EN 12697. Three replicates of each mixture were rotated with a stirrer in the bottle for 72 hours with 60 rotations per minute and note reading after 6, 24, 48, and 72h. The results are discussed in Figure 8.
Figure 8 shows the evaluation of % loss of bitumen covering from aggregate at standard laboratory test conditions. The presence of substitute fillers in asphalt mixture significantly reduces the % loss of aggregate-bitumen cover and improves the adhesion properties of the mixture. Both cement and hydrated lime as substitution filler improves the adhesion of the mixtures. Contrary to previous results, cement gives better results as compared to hydrated lime for adhesion properties of the mixtures.

3.5. Summary and Discussion

The overall trends of substitution fillers on the performance of asphalt mixture are summarized in Table 8. The table was generated by using the experimental test results and the following thoughts: “what is the most likely trend obtained from performance-based test results?” The size of the arrows indicates the importance of parameter on the performance of asphalt mixture. Larger size arrow indicates higher impact and smaller arrow size indicate the lower impact of substitute filler on HMA performance.

![Figure 8. Effect of moisture after 72h](image)

Table 8. Performance based indication of asphalt mixture

| Mixture Type                                      | Rutting resistance | Fatigue resistance | Dynamic modulus | Moisture susceptibility |
|--------------------------------------------------|--------------------|--------------------|-----------------|------------------------|
| Calcium carbonate minerals with cement filler    | ↑                  | ↑                  | ↑               | ↑                      |
| Calcium carbonate minerals with hydrated lime filler | ↑                  | ↑                  | ↑               | ↑                      |
| Dolerite minerals with cement filler             | ↑                  | ↑                  | ↑               | ↑                      |
| Dolerite minerals with hydrated lime filler      | ↑                  | ↑                  | ↑               | ↑                      |

Legend:

More beneficial effect = ↑

Less beneficial effect = 

The overall performance of substitute fillers in Table 8 shows that all the mixes improve rutting and fatigue resistance, visco-elastic behavior, and resistance against moisture susceptibility from its conventional mixes. Hydrated lime as a substitute filler has more beneficial for rutting and fatigue resistance as well as for visco-elastic behavior. Similarly, cement as a substitute filler in the mixes has more favorable for resistance against moisture damage.
4. Conclusion

Overall, in this research study, two substitution fillers (cement & lime) were used with two different aggregate quarries (based on minerals composition) to evaluate the relatively most beneficial combination of both fillers as well as an aggregate quarry to enhance the performance life of asphalt pavements, especially in under-developed countries. Four basic tests (Asphalt Pavement Analyzer, Four Points Bending Beam, Dynamic Modulus, and Rolling Bottle Test) that test the most desired properties of any asphalt pavement were utilized to access the performance properties of modified asphalt mixture. Based on all laboratory test results, it is concluded that replacement of filler with cement and hydrated lime improves the fatigue, adhesion, and rut performance of asphaltic concrete mixture up to 25%, 70%, and 25% respectively to that of the conventional mixture.

However, cement as substitution filler in asphalt mixture has a comparatively more beneficial effect on permanent deformation and fatigue performance in comparison with lime. While cement substitute (as filler) enhances the adhesion properties (moisture resistance) of asphalt mixture. Cement has a relatively stronger bond with calcium carbonate aggregate quarry, while lime substitute filler has a more compatible bond with dolerite type aggregate quarry.

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6. Conflicts of Interest

The authors declare no conflict of interest.

7. References

[1] Guha, Aioub H., and Gabriel J. Assaf. “Effect of Portland Cement as a Filler in Hot-Mix Asphalt in Hot Regions.” Journal of Building Engineering 28 (March 2020): 101036. doi:10.1016/j.jobe.2019.101036.

[2] Bahia, Hussain U., D. I. Hanson, M. Zeng, H. Zhai, M. A. Khatri, and R. M. Anderson. “Characterization of modified asphalt binders in superpave mix design.” No. Project 9-10 FY’96. (2001).

[3] Al-Suhaibani, A, J Al-Mudaheem, and F Al-FOzan. “Effect of Filler Type and Content on Properties of Asphalt Concrete Mixes.” Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance (1992): 107–124. doi:10.1520/stp24214s.

[4] Lesueur, Didier, Joëlle Petit, and Hans-Josef Ritter. “The Mechanisms of Hydrated Lime Modification of Asphalt Mixtures: a State-of-the-Art Review.” Road Materials and Pavement Design 14, no. 1 (December 5, 2012): 1–16. doi:10.1080/14680629.2012.743669.

[5] Xu, Ouming, Zhenjun Wang, and Rui Wang. “Effects of Aggregate Gradations and Binder Contents on Engineering Properties of Cement Emulsified Asphalt Mixtures.” Construction and Building Materials 135 (March 2017): 632–640. doi:10.1016/j.conbuildmat.2016.12.095.

[6] Ramaswamy, Salem D., and M. A. Aziz. "Assessment of Effect of Filler on the Stripping of Bituminous Mix." (1983).

[7] Ramaswamy, S. D., and M. A. Aziz. "Effect of Filler Type and Shape of Aggregates on the Stability of Bituminous Mixes." (1983).

[8] Wang, Jie, Meng Guo, and Yiqiu Tan. “Study on Application of Cement Substituting Mineral Fillers in Asphalt Mixture.” International Journal of Transportation Science and Technology 7, no. 3 (September 2018): 189–198. doi:10.1016/j.ijtst.2018.06.002.

[9] Xiao, Jingjing, Wei Jiang, Wanli Ye, Jinhuan Shan, and Zhenjun Wang. “Effect of Cement and Emulsified Asphalt Contents on the Performance of Cement-Emulsified Asphalt Mixture.” Construction and Building Materials 220 (September 2019): 577–586. doi:10.1016/j.conbuildmat.2019.06.051.

[10] Zhou, Bin, James Bentham, Mariachiara Di Cesare, Honor Bixby, Goodarz Danaei, Melanie J Cowan, Christopher J Paciorek, et al. “Worldwide Trends in Blood Pressure from 1975 to 2015: a Pooled Analysis of 1479 Population-Based Measurement Studies with 19·1 Million Participants.” The Lancet 389, no. 10064 (January 2017): 37–55. doi:10.1016/s0140-6736(16)31919-5.

[11] Huang, Shin-Che, Raymond E. Robertson, Jan F. Branthaver, and J. Claine Petersen. "Impact of lime modification of asphalt and freeze–thaw cycling on the asphalt–aggregate interaction and moisture resistance to moisture damage." Journal of materials in civil engineering 17, no. 6 (2005): 711-718. doi:10.1061/(ASCE)0899-1561(2005)17:6(711).

[12] Kakade, Vijay B., M. Amaranantha Reddy, and K. Sudhakar Reddy. “Evaluation of the Sensitivity of Different Indices to the Moisture Resistance of Bituminous Mixes Modified by Hydrated Lime and Other Modifiers.” Road Materials and Pavement Design 18, no. 6 (September 1, 2016): 1395–1410. doi:10.1080/14680629.2016.1224198.
[13] Kakade, Vijay B., M. Amaranatha Reddy, and K. Sudhakar Reddy. “Effect of Aging on Fatigue Performance of Hydrated Lime Modified Bituminous Mixes.” Construction and Building Materials 113 (June 2016): 1034–1043. doi:10.1016/j.conbuildmat.2016.03.066.

[14] Rasouli, Amir, Amir Kavussi, Mortezza Jali Jazizadeh, and Amir Hossein Taghkhani. “Evaluating the Effect of Laboratory Aging on Fatigue Behavior of Asphalt Mixtures Containing Hydrated Lime.” Construction and Building Materials 164 (March 2018): 655–662. doi:10.1016/j.conbuildmat.2018.01.003.

[15] Atud, Tebid Joshua, Kunnawee Kanitpong, and Wilfing Martono. “Laboratory evaluation of hydrated lime application process in asphalt mixture for moisture damage and rutting resistance.” No. 07-1508. (2007).

[16] Khattak, Mohammad J., and Vikram Kyatham. “Viscoelastic Behavior of Hydrated Lime-Modified Asphalt Matrix and Hot-Mix Asphalt Under Moisture Damage Conditions.” Transportation Research Record: Journal of the Transportation Research Board 2057, no. 1 (January 2008): 64–74. doi:10.3141/2057-08.

[17] Mohammad, Louay N., Shadi Saadeh, Md Kabir, Ayman Othman, and Sam Cooper. “Mechanistic Properties of Hot-Mix Asphalt Mixtures Containing Hydrated Lime.” Transportation Research Record: Journal of the Transportation Research Board 2051, no. 1 (January 2008): 49–63. doi:10.3141/2051-07.

[18] Petersen, J. Claine, Henry Plancher, and P. Michael Harnsberger. "Lime treatment of asphalt to reduce age hardening and improve flow properties." In Association of Asphalt Paving Technologists Proceedings Technical Sessions, 1987, Reno, Nevada, USA, vol. 56. (1987).

[19] Lime, Hydrated. "A proven additive for durable asphalt pavements." European Lime Association (2010).

[20] Bari, Javed, and Matthew W. Witzczak. “Evaluation of the Effect of Lime Modification on the Dynamic Modulus Stiffness of Hot-Mix Asphalt.” Transportation Research Record: Journal of the Transportation Research Board 1929, no. 1 (January 2005): 10–19. doi:10.1177/0361198105192900102.

[21] Aragão, Francisco Thiago Sacramento, Junghun Lee, Yong-Rak Kim, and Pravat Karki. “Material-Specific Effects of Hydrated Lime on the Properties and Performance Behavior of Asphalt Mixtures and Asphaltic Pavements.” Construction and Building Materials 24, no. 4 (April 2010): 538–544. doi:10.1016/j.conbuildmat.2009.10.005.

[22] Berthelot, Curtis, Ania Anthony, Colin Wandzura, and Brent Marjerison. “Triaxial frequency sweep characterization of asphalt-aggregate adhesion in Saskatchewan asphaltic mixes.” Journal of transportation engineering 136, no. 2 (2010): 158–164. doi:10.1061/(ASCE)0733-947X(2010)136:2(158).

[23] Özen, Halit. “Rutting Evaluation of Hydrated Lime and SBS Modified Asphalt Mixtures for Laboratory and Field Compacted Samples.” Construction and Building Materials 25, no. 2 (February 2011): 756–765. doi:10.1016/j.conbuildmat.2010.07.010.

[24] Sengül, Celaleddin E., Atakan Aksoy, Erol Iskender, and Halit Özen. “Hydrated Lime Treatment of Asphalt Concrete to Increase Permanent Deformation Resistance.” Construction and Building Materials 30 (May 2012): 139–148. doi:10.1016/j.conbuildmat.2011.12.031.

[25] Souliman, Mena I., Murugaiyah Piratheepan, Elie Y. Hajj, Peter E. Sebaaly, and Wendy Sequeira. “Impact of Lime on the Mechanical and Mechanistic Performance of Hot Mixed Asphalt Mixes.” Road Materials and Pavement Design 16, no. 2 (March 5, 2015): 421–444. doi:10.1080/14680629.2015.1017520.

[26] Roy, Neetu, A. Veeraragavan, and J. Murali Krishnan. “Influence of Air Voids of Hot Mix Asphalt on Rutting Within the Framework of Mechanistic-Empirical Pavement Design.” Procedia - Social and Behavioral Sciences 104 (December 2013): 99–108. doi:10.1016/j.sbspro.2013.11.102.

[27] Vansteenkiste, Stefan, Joëlle De Visscher, and Ann Vanelstraete. “Impact of Hydrated Lime on the Durability of SMA Mixtures: Laboratory and Field Evaluation.” Road Materials and Pavement Design 14, no. sup1 (April 2013): 162–174. doi:10.1080/14680629.2013.774753.

[28] Zalkati, Anggraini, Wong Yiik Diew, and Darren Sun Delai. “Effects of Fillers on Properties of Asphalt-Concrete Mixture.” Journal of Transportation Engineering 138, no. 7 (July 2012): 902–910. doi:10.1061/(asce)te.1943-5436.0000395.

[29] Chompoorat, T., and S. Likitlersuang. “Laboratory investigation of hot mix asphalt behaviour for mechanistic-empirical pavement design in tropical countries.” Geotechnical Engineering Journal of the SEAGS & AGSSEA 46, no. 1 (2015): 37–44.

[30] Christensen, Donald W., and David A. Anderson. "Interpretation of dynamic mechanical test data for paving grade asphalt cements (with discussion)." Journal of the Association of Asphalt Paving Technologists 61 (1992).
Climate Change Scenarios and Effects on Snow-Melt Runoff

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Abstract

Climate change is an important environmental issue, as progression of melting glaciers and snow cover is sensitive to climate alteration. The aim of this research was to model climate alterations forecasts, and to assess potential changes in snow cover and snow-melt runoff under the different climate change scenarios in the case study of the Zayandeh-rud River Basin. Three cluster models for climate change (NorESM1-M, IPSL-CM5A-LR and CSIRO-MK3.6.0) were applied under RCP 8.5, 4.5 and 2.6 scenarios, to examine climate influences on precipitation and temperature in the basin. Temperature and precipitation were determined for all three scenarios for four periods of 2021-2030, 2031-2040, 2041-2050 and 2051-2060. MODIS (MOD10A1) was also applied to examine snow cover using temperature and precipitation data. The relationship between snow-covered area, temperature and precipitation was used to forecast future snow cover. For modeling future snow melt runoff, a hydrologic model of SRM was used including input data of precipitation, temperature and snow cover. The results indicated that all three RCP scenarios lead to an increase in temperature, and reduction in precipitation and snow cover. The results indicated that all three RCP scenarios lead to an increase in temperature, and reduction in precipitation and snow cover. Investigation in snowmelt runoff throughout the observation period (November 1970 to May 2006) showed that most of annual runoff is derived from snow melting. Maximum snowmelt runoff is generated in winter. The share of melt water in the autumn and spring runoff is estimated at 35 and 53%, respectively. The results of this study can assist water manager in making better decisions for future water supply.

Keywords: Climate Change; Snow Cover; Snow Melt; SRM Model; RCP Scenarios.

1. Introduction

Rainfall pattern is changing in terms of volume, intensity and form around the world [1]. Snow coverage and continuity depend on the amount of precipitation and temperature, which are strongly related to climatic condition [2]. If the area of snow cover changes, future access to and management of water could be difficult in the long-term [3], because the amount of runoff and flooding are likely to increase. Since 2001 onwards, the importance of climate change for hydrologic systems which are affected by snow has been studied. Aziz et al. (2020) [4] showed that slight increases in temperature can significantly affect timing in runoff events in mountainous areas. Runoff is usually increasing in cold season and decrease e during warm period, probably because of snow melt as a result of increased temperature. Emmer et al. (2019) [5] has evaluated future runoff from three glacial areas in Peru using a hydrologic model, which included changes in snow cover. They found that the total volume of the area’s glaciers decreased by 78% between 1971 and 2015 (because of increased evapotranspiration), furthermore by 2100, under scenarios A2 and B2 the area will have no glaciers coverage. Aili et al. (2019) [6] found that up to 2100, between around 3.9 and 6.8 km of glaciers would be lost in the Rio Maipo area, in Chile.
Studies have been also conducted in glacial basins in Asia, by for example Zhang et al. (2016) and Treichler et al. (2019) [7, 8]. The results showed that homogeneous regional variations in the Northern and Southern sites of the mountain are likely to have happened, and glacial cover is lost. Access to water in the Himalayas under climate change conditions was studied by Momblanch et al. (2019) [9]. They showed that future snowmelt runoff in different areas in the Himalayas can generate from melting between 100 or 50 percentage of glacier until 2100. The snowmelt runoff created because of temperature rise (rise by between 4 and 5 °C) in the future, and because of precipitation has risen by 7% in the period up until 2100. Liu et al. (2020) [10] showed that as little as 8% of the river basin glaciers in Central Asia might remain under scenarios A2 and B2, by the end of the 21st century, so most of the river basin glaciers are predicted to be lost by them.

The effect of climate change on streamflow is analyzed in previous researches. Hydrologic modelling is a valuable tool for flood forecasting and decision making in water resources organization [11]. Previously, various hydrologic models with a snow factor were applied to model the daily stream flows in snow- and glacier-fed catchments [12-14]. Nevertheless, most of the hydrologic models are not practical to use for daily stream flow simulation and projection in the mountainous catchments, where the snowmelt is a main parameter in the water cycle [15, 16]. Because many of these models are sensitive to the precipitation driving, the precipitation data available from the high altitude catchments is usually not of very decent quality [17]. Lack of information for temporal and spatial rainfall variability causes a huge uncertainty in snowmelt runoff forecasting [18, 19].

The effects of climate change on climatic conditions and surface water resources in Iran have also been investigated. Yoosof Doost et al. (2018) [20] used the HADCM3 climate model under the scenarios A2 and B2 to study the effects of climate alteration in the Taleghan Basin in Iran. More details about HADCM3 climate model under scenarios A2 and B2 is shown in Valdes et al. (2017) study [21]. However, there are few studies that have analyzed the effects of climate change on snowmelt runoff in Iran.

Since a significant proportion of precipitation in the Zayandeh-rud area is snow, thus snowmelt water plays a major role in the River water supply. Because there is a lack of data and numerical values for snowmelt runoff, moreover because of lack of control on the snowmelt runoff, therefore investigating the value of snowmelt runoff under the climate change effects is very important. Understanding the relationship between climate change, and snow and ice runoff is essential in water management. The purpose of this study was to investigate how future temperature might affect snow cover and snow melt in the Zayandeh-rud area. Unlike previous studies (e.g., [22]), the data presented in the fifth report (CMIP5) were considered by simulating climate change scenarios in this study. More details about CMIPS is presented in Salman et al. (2018) research [23]. In this study, the structure of climate change models and scenarios to project the future climate change effects on temperature and precipitation is given. This is including the description of the snow-melt runoff model and snow cover area, which are explained in section 2. In section 3, the results of snowfall projection, snow coverage areas and snow melting forecasts are presented. The comparison of the results with the previous studies, are explained in section 4 and the summary of new findings including the projection of seasonal climate change effects on snow melt parameters is presented in section 5.

2. Materials and Methods

2.1. Study Area

The study area is one of the main basins in the Kafrud desert, with an area of 41,548 km², between 32°10’ and 33°40’ N, and 50°30’ and 53°23’ E. It is bounded to the north by the “Salt Lake” (which is small), to the west by the Gulf of Oman and the Oman Sea, and to the east by the Kavir-siyahkoooh mountain range to the south of the Kavir Sirjan sub zone (which is located in the south of Zayandeh Rud basin). Its important rivers include the Zayandeh-rud (405 km long), the Khoshkehrood (165 km), and the Izodkhad (125 km). The catchment covers parts of Isfahan, Chaharmahal and Bakhtiari, Fars and Yazd provinces, with Isfahan Province accounting for more than 83% and Yazd province by less than 3.5%. Figure 1 illustrates the study area.

Natural flows in the Zayandeh-rud River are increased by the diversion of water through the first and second tunnels of Koohrang, originating from Koohrang River in Chaharmahal and Bakhtiari Province. Because the average rainfall in the basin is less than 150 mm/year, Zayandeh-rud Dam, in Chadegan region, stores spring and winter runoff, which then is released to the main River. The upstream parts of the basin comprise less than 10% of the whole catchment and are mostly mountainous. The central and lower parts of the basin (89%) are sedimentary plains, and are used for agriculture. Many overflows and detours have been constructed along the river, to take water for urban and industrial usage. The Zayandeh-rud Basin and water flows end in natural swamp of Gavkhoni and into the seasonal salt marshlands.

In this study, meteorological data were used to forecast the area’s possible future climate, and also the data from hydrometric stations and statistics to simulate runoff. The availability of historic data records with lower statistical errors were considered in this study as main criteria.
The flowchart of overall research methodology is shown in Figure 2.

- **Climate change models and scenarios**
  - NorESM1-M, IPSL-CM5A-LR and CSIRO-MK3.6.0 under RCP 8.5, 4.5 and 2.6 scenarios

- **Future projection of climate change effects on temperature, precipitation and snowfall**

- **Modeling the snow cover area by using input data from snowfall and get the output from MODIS (MOD10A1)**

- **Modeling snow melt runoff by using input data from climate change model, snow cover area and to get the output of SRM**

Figure 2. The structure of the methodology in this study
Snow cover data were extracted from Moderate Resolution Imaging Spectroradiometer (MODIS) satellite and daily snow cover maps, (which used from MODIS are named MOD 10A1) for this area. Snow precipitation was modeled using the snowmelt runoff model (SRM) [25] and verified with historic data. Climate change scenarios were extracted using general flow and downscaled models (e.g., the delta change method [26]), applying statistical methods based on the delta change method for the study area, and their effect on snowmelt runoff under these scenarios was investigated.

2.2. Snow Cover

Determining the level of snow cover in the simulation of snowmelt runoff in this area is of particular importance. Some 1,450 daily snow cover images from MODIS (MOD 10A1) were used from the period between 2000 to 2007 determine the extent of the cover in this area.

2.3. Climate Change Projection

The models presented in CMIP5 have better spatial quality than those in CMIP3, so in this research different General Circulation Models (GCMs) from CMIP5 were used. The images were entered into ARCGIS 10/1 software and the daily proportion of snow cover was calculated [27]. Cloud cover is an obstacle in determining the true extent of snow cover, thus images with cloud cover exceeding 20% were eliminated, and average snow cover was considered based on linear interpolation between the days immediately before and after.

2.4. Snow-melt Runoff Model

The snow-melt runoff model is one of the most widely used for simulating daily flows in mountainous areas and has been successfully tested by the World Meteorological Organization. It is a conceptual hydrologic model, and can be used to simulate daily runoff, and to predict snow–melt and precipitation. In this study, simulated daily runoff from snow-melt and rainfall in the study area were calculated using Equation 1.

\[ Q_{n+1} = C_{sn} \ a_n \ (T_{n+1} + \Delta T_n) \ s_n + A \cdot 0.116 \ (1 + K_{n+1}) + C_{rn} P_n + A \cdot 0.116 \ (1 - K_{n+1} + (Q_{Sn} + Q_{rn}) K_{n+1}) \]  

(1)

Where \( Q \) is the daily mean discharge (m^3/s), \( C_s \) the coefficient of snow runoff, \( C_r \) the coefficient of rainfall runoff, “\( a_n \)” is a coefficient for degree-days (cm C^-d^-1), \( T + \Delta T \) the number of degrees and days (cd); \( S \) the ratio of snow cover to the total area, \( P \) represents daily rainfall (cm), \( A \) the basin area (km^2), \( K \) a reduction factor (Y_c, X_c), and \( n \) the number of days simulated. The factor 0.116 is a constant.

The 38 models of CMIP5 were implemented with new scenarios (e.g., Representative Concentration Pathway (RCP)). RCP scenarios represent radiative drives. A Representative Concentration Pathway (RCP) is a greenhouse gas concentration (not emissions) trajectory, adopted by the IPCC for its fifth Assessment Report (AR5) in 2014. It supersedes Special Report on Emissions Scenarios (SRES) projections published in 2000.

Three pathways have been selected for climate modeling and research, which describe different climate futures, all of which are considered as possible, depending on how much greenhouse gases are emitted in the years to come. The three RCPs, namely RCP2.6, RCP4.5, and RCP8.5, are labelled after a possible range of radiative forcing values in the year 2100. Among all scenarios, RCP 2.6 are the scenarios with low radiation propulsion patterns and consist B1 (convergence and ecologically friendly) and B2 scenarios [28] and the scenarios of RCP 8.5 show a scenario of radiative drives with high radiation propulsion patterns.

Three models, NorESM1-M, IPSL-CM5A-LR and CSIRO-MK3.6.0, were selected and used under scenarios RCP 2.6, 4.5 and 8.5 for four periods – 2021-2030, 2031-2040, 2041-2050 and 2051-2060 – to try to predict the greatest, least and most intermittent changes in climate. In this study, the three models which have been modelled showed the best fit with observed climate parameters (for example precipitation modelled by these three models is similar to the precipitation which is estimated by rain gauges for historical time period). NorESM1-M was selected as an exponential model for model calibration. In this study, among the three models, the model of NorESM1-M showed the best simulation and therefore best fit with observed climate parameters.

2.5. Estimating Future Snow Cover

Snow cover is an important parameter in the area’s runoff pattern. Water storage in the form of snow and ice helps modify runoff variability, created by the rainfall pattern. Changes in snow cover area cause changes in snow-melt runoff. For this study, future snow coverage was estimated using the relationship between the snow-covered area, temperature and rainfall. Multivariate analysis based on regression using SPSS was performed with 4 predictor variables – mean monthly temperature, mean monthly precipitation, and the previous month’s temperature and precipitation – to calculate snow cover at altitudes above 1,800 m.
3. Results

3.1. Analysis of Snow Covers Area

Snow cover in the area starts in November and is most extensive in January, which is a very cold month. After that, the area covered decreases because of increasing temperature and evapotranspiration. In June, the area covered by snow is at its lowest, because the temperature and evapotranspiration are both at their maximum. Figure 3 shows the monthly variation in snow cover at different altitudes (period 2000 to 2006). The extent of snow cover increases with topographic level. As Figure 3 indicates the snow cover for each month and for each year fluctuates.

![Figure 3. Proportional snow covers at different elevations – monthly, 2000 to 2006](image)

3.2. Runoff Simulation

The runoff model was calibrated and evaluated for the period 1973 to 2006. Figure 4 shows the simulated and measured discharge variations. Because snow cover area changes a lot (as showed in Figure 3), so the runoff also has a fluctuation for each year. The coefficient of determination ($R^2$) is 0.70 and the percentage difference between the estimated and observed runoff is 35%. The quality of the simulation is thus acceptable. The difference is because the model cannot calculate the water seepages and water losses.

![Figure 4. Comparison of observed and simulated discharges - The Zayandeh-rud basin [24]](image)

The proportion of snow-melt in relation to a total runoff from the area for the period between 2000 to 2006 was investigated, and the results indicate that maximum snow-melt occurs in winter and spring (55%). Snow-melt runoff is highest in winter because, especially in March (late winter), the temperature rises and the speed of melting increases.
Table 1. Annual average snow-melt proportion in total runoff – 2000 to 2006

| Season | Average snow-melt component (cm) | Total runoff generated (cm) |
|--------|----------------------------------|----------------------------|
| Fall   | 34                               | 96                         |
| Winter | 75                               | 140                        |
| Spring | 53                               | 100                        |
| Summer | 160                              | 400                        |

3.3. Precipitation and Temperature Forecasting

The three climate models of NorESM1-M, IPSL-CM5A-LR and CSIRO-MK3.6.0, were used to predict the temperature and precipitation under the three RCP scenarios. The model was calibrated using historical data from 2000 to 2006, and showed a high correlation between observed and simulated temperature and precipitation. In general, the model can simulate climate parameters very well.

Table 2. Correlation coefficients between observed and simulated temperature and precipitation obtained using SPSS

| Parameter       | Correlation coefficient |
|-----------------|-------------------------|
| Temperature     | 0.80                    |
| Precipitation   | 0.62                    |

The average temperature and precipitation for the period November to June were calculated, and the downscaled output of NorESM1-M, IPSL-CM5A-LR and CSIRO-MK3.6.0 was calculated according to Table 3. The grid was 1.5×1.5 km and some 18,222 grids were used. The downscaled output was used because the data from the climate model were at large scale in order to obtain data at the scale of the study area [29].

The simulation shows that the average temperature increases with respect to the base period. This means that in coming decades, there is an average increase in the different scenarios, for example the average temperature is increased between 1.1 and 3.4 °C in NorESM1-M, between 2 and 4.35 °C in IPSL-CM5A-LR, and between 1.3 and 3.5 °C in CSIRO-MK3.6.0.

Comparing the predicted temperatures in the models, the highest temperatures are found in the 2050s in IPSL-CM5A-LR under scenario RCP 8.5. The average temperature increases in the 2050s is about 1.8, 2.8 and 3.5 °C, respectively, for RCP scenarios 2.6, 4.5 and 8.5. The trend of annual average changes for November to June indicates that temperature increases by about 0.05 °C/a. The results of the study also indicate decreasing trend in annual precipitation compared to the base period. NorESM1-M indicates a greater reduction in precipitation than the other two models. The three climate models suggest that the mean rainfall in the 2050s will be 2, 3.2, and 4.5% lower for RCP scenarios 2.6, 4.5 and 8.5, respectively. The greatest reduction indicated in the 2050s is about 14 mm for RCP scenario 8.5. The average annual rainfall from November to June drops about 0.038%/a.

Table 3. Temperature and precipitation in the period November to June for different models and scenarios

| Parameter                  | Time period | NorESM1-M       | IPSL-CM5A-LR    | CSIRO-MK3.6.0 |
|----------------------------|-------------|-----------------|-----------------|---------------|
|                            |             | RCP 2.6         | RCP 4.5         | RCP 8.5       | RCP 2.6         | RCP 4.5         | RCP 8.5       | RCP 2.6         | RCP 4.5         | RCP 8.5       |
| Mean annual temperature    | 2021-30     | 9.1             | 9.31            | 9.41           | 9.71           | 9.83           | 9.90           | 9.51           | 9.37           | 9.51           |
| (°C)                       | 2031-40     | 9.3             | 9.57            | 9.8            | 9.99           | 10             | 10.50          | 9.90           | 9.90           | 9.97           |
|                            | 2041-50     | 9.6             | 10              | 10.3           | 10.17          | 10.59          | 10.97          | 9.30           | 10.20          | 10.57          |
|                            | 2051-60     | 9.8             | 10.13           | 10.48          | 10.17          | 11.28          | 11.12          | 9.30           | 10.40          | 10.69          |
| Mean annual precipitation  | 2021-30     | 336.2           | 311.24          | 317.20         | 297.12         | 299.17         | 299.80         | 303            | 300.7          | 298.20         |
| (mm)                       | 2031-40     | 339.24          | 324.7           | 322.10         | 298.09         | 296.78         | 294.03         | 295            | 293            | 298.78         |
|                            | 2041-50     | 337.14          | 327.10          | 322.03         | 298.09         | 294.89         | 295            | 297            | 296            | 295.80         |
|                            | 2051-60     | 332.08          | 319.19          | 316.15         | 298.20         | 291            | 292.20         | 293.5          | 290            | 289            |

3.4. Forecasting Runoff

After calibration, the SRM model was used to simulate runoff using different GCM and RCP outputs for the study periods. Table 4 shows the average runoff in the simulated periods.
Table 4. Runoff from November to June for different models and scenarios

| Parameter (m³/s) | Time period | NorESM1-M | IPSL-CM5A-LR | CSIRO-MK3.6.0 |
|-----------------|-------------|-----------|--------------|---------------|
|                 | RCP 8.5     | RCP 4.5   | RCP 2.6      | RCP 8.5       | RCP 4.5       | RCP 2.6       | RCP 8.5       | RCP 4.5       | RCP 2.6       |
| Runoff          | 2021-30     | 157       | 143          | 150           | 120.50        | 123           | 118.20        | 123.45        | 125           | 120           |
|                 | 2031-40     | 152       | 142          | 154           | 119           | 120           | 118           | 123.50        | 122.20        | 123.20        |
|                 | 2041-50     | 154       | 145          | 154.70        | 115.75        | 114           | 115           | 117           | 116           | 115.1         |
|                 | 2051-60     | 151.50    | 141.50       | 143.1         | 115           | 96            | 98            | 120           | 118.8         | 100.01        |

The average runoff rate is reduced by about 0.2 m³/s per year from the year of between 2006 to 2100. With increasing temperature and decreasing rainfall, the amount of snow cover decreases, reducing the volume of snow-melt related runoff. Total annual runoff volume is expected to decrease by about 12% (RCP 2.6), 13% (4.5), and 10% (8.5). The reduction in runoff in RCP scenario 8.5 is probably less than in the other two because of increased rainfall in the fall.

3.5. Future Snow-melt

The snow-melt proportion predicted in relation to the total runoff in the Zayandeh-rud River Basin is significantly reduced, from about 40% historically to 29% in decade 2051-60, because of a combination of reduced precipitation and some change from snow to rain. All models predict reduced snow-melt volumes, by about 28% on average. Figure 5 indicates the results of forecasting for snow melt. As figure presents, the results of different climate change scenarios are different. As shown in the figure, the proportional contribution of snow-melt to runoff is reduced in all scenarios, and most significantly in spring.

3.6. Snowfall Forecasting

Figure 6 shows the Pearson correlation coefficient for snow cover with temperature and precipitation in the area. The results show that snow melt values follow the amount of precipitation and temperature. Temperature has a negative relationship with snow cover because if temperature increases, then the area of snow cover will decrease. Precipitation in May and June has a negative relationship with snow cover, because at this time, precipitation is in form of rain instead of snow. Snow fall from December through April can increase snow cover, because at this time, precipitation is in form of snow and there is a positive relationship with snow cover. Therefore between March and April the highest correlation between precipitation and snow cover is observed and between May and June there is least correlation between temperature and snow cover. That is because of the temperature increases, and the snow cover decreases.

![Figure 5. Proportion of snow-melt to total annual average runoff for observed data and three RCP scenarios for duration of 2006 to 2100](image-url)
The relationship between snow cover, temperature and precipitation was analyzed using the temperature and precipitation projections, and the extent of snow cover estimated for the years 2021 to 2060. Annual snow cover is more likely to decrease 12% by 2020, 17% by 2030, 20% by 2040, and 22% by 2050, compared with the historic period. The largest reduction in snow cover is observed annually for the decade from 2050 in model IPSL-CM5A-LR under scenario RCP 8.5, and the lowest in model NorESM1-M under scenario RCP 2.6 in the decade from 2020. Table 5 shows snow cover predictions for the different models and scenarios.

| Parameter | Time period | NorESM1-M | IPSL-CM5A-LR | CSIRO-MK3.6.0 |
|-----------|-------------|-----------|--------------|---------------|
|           | RCP 2.6 | RCP 4.5 | RCP 8.5 | RCP 2.6 | RCP 4.5 | RCP 8.5 | RCP 4.5 | RCP 2.6 |
| Area of snow cover (km²) | 2021-30 | 2007 | 1797 | 1780 | 1769 | 1772 | 1775 | 1779 | 1790 | 1780 |
|          | 2031-40 | 1792 | 1745 | 1722 | 1715 | 1716 | 1700 | 1780 | 1692 | 1773 |
|          | 2041-50 | 1693 | 1750 | 1625 | 1650 | 1640 | 1570 | 1692 | 1800 | 1960 |
|          | 2051-60 | 1650 | 1700 | 1555 | 1600 | 1557 | 1480 | 1693 | 1670 | 1599 |

4. Discussion

Hydrographic change forecasts are important for water resource management. Previous studies did not use the current climate models and did not attempt to forecast snow-melt runoff. Equally, they did not compare the value of snow melt runoff between current and future time period. This study provides a range of climate change predictions for future snow-melt runoff yield in the Zayandeh-rud River Basin between 2021 and 2060. The latest climate models (CMIP5) were used under the RCP 2.6, 4.5 and 8.5 scenarios. Snow cover images were used to determine the snow cover area in high altitude areas of Zagros, in places where there are relatively few snowfall stations. Images from the Zayandeh-rud Basin show extensive snow cover at altitudes above 1,800 m. snow cover is high (above 3,000 m ) from December to February, but snow cover is low in May and June (less than 3,000 m).

The SRM’s efficiency in mountainous catchments can be credited to the use of MODIS, which remotely sensed snow cover data as an input into the model. However, as Sharma et al. (2020) [30] mentioned still more research is needed to do this analysis. As Hao et al. (2019) [31] mentioned the analysis of the climate change impact indicated that watershed hydrology would alter under different climate change scenarios. In simulating snow-melt with the SRM model, validation indicates that the model's performance is acceptable in estimating runoff in the study area. In comparison with the observed historical period (2000 to 2007), the value of snow-melt runoff in fall and winter could be reduced by 10% and 24% in the year of 2060.
The indications are that the mean annual temperature in coming decades will increase. The results obtained and arising from temperature changes in the different scenarios, are consistent with those obtained by others recently studied [32-34].

The study’s results indicate that total annual rainfall in the study area will decrease over the period up to 2060. The temperature is more likely to increase and snow areas, and snow-melt and runoff are likely to increase. The results of this study, based on reductions in runoff and snow-melt, are consistent with those of other researchers [35-37]. Also consistent to previous studies that indicate the increased/decreased snow cover area results in the increasing/decreasing in total flow and pick flow during different season e.g. [38-40].

5. Conclusion

This research has used integrated models to model the future snow cover and snow melt under different climate change scenarios. The study forecasted climate alterations, and assessed the potential changes in snow cover and snow-melt runoff under different climate change scenarios in the Zayandeh-rud Basin. Three climate change models of cluster models (NorESM1-M, IPSL-CM5A-LR and CSIRO-MK3.6.0) applied under RCP 8.5, 4.5 and 2.6 scenarios, to examine climate influences the precipitation and temperature in the basin. Temperature and precipitation determined for all three scenarios for four periods –2021-2030, 2031-2040, 2041-2050 and 2051-2060. MODIS (MOD10A1) applied to examine snow cover. The relationship between snow-covered area, and temperature and precipitation used to forecast future snow cover. By inputting climate data into the SRM model which used climate data as an input to analyze the impact of climate change on the hydrological process. The analysis of the climate change impact indicated that watershed hydrology would alter under various time periods and different seasons. The results indicated that all three RCP scenarios will lead to an increase in temperature, and reduction in precipitation and snow cover. Investigation of snowmelt runoff throughout the observation period (November 1970 to May 2006) showed that most of annual runoff is derived from melting snow. Maximum snowmelt runoff is generated in winter. The share of meltwater in the autumn and spring runoff is estimated at 35 and 53%, respectively. In addition, the study results indicate that the total annual rainfall in the study area will decrease over the period up to 2060. The temperature is more likely to increase and snow areas, and snow-melt and runoff are likely to increase. The results of this study, based on reductions in runoff and snow-melt, are consistent with those of other researchers. However, the obtained modeling results will offer valuable decision support mechanism for water resources management and for development of local ecosystem sustainability and social-economic improvements.

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7. Conflicts of Interest

The authors declare no conflict of interest.

8. References

[1] Zarenistanak, Mohammad. “Historical Trend Analysis and Future Projections of Precipitation from CMIP5 Models in the Alborz Mountain Area, Iran.” Meteorology and Atmospheric Physics 131, no. 5 (September 1, 2018): 1259–1280. doi:10.1007/s00703-018-0636-z.

[2] Ansari, Hadi, Safar Marofi, and Mohamad Mohamadi. “Topography and Land Cover Effects on Snow Water Equivalent Estimation Using AMSR-E and GLDAS Data.” Water Resources Management 33, no. 5 (February 11, 2019): 1699–1715. doi:10.1007/s11269-019-2200-0.

[3] Butt, Mohsin Jamil, Mazen Ebraheem Assiri, and Ahmed Waqas. “Spectral Albedo Estimation of Snow Covers in Pakistan Using Landsat Data.” Earth Systems and Environment 3, no. 2 (June 18, 2019): 267–276. doi:10.1007/s41748-019-00104-1.

[4] Aziz, Rizwan, Ismail Yucel, and Ceylan Yozgatgil. “Nonstationarity Impacts on Frequency Analysis of Yearly and Seasonal Extreme Temperature in Turkey.” Atmospheric Research 238 (July 2020): 104875. doi:10.1016/j.atmosres.2020.104875.

[5] Emmer, Adam, Anna Juřicová, and Bijeesh Kozhikkodan Veettil. “Glacier Retreat, Rock Weathering and the Growth of Lichens in the Churup Valley, Peruvian Tropical Andes.” Journal of Mountain Science 16, no. 7 (July 2019): 1485–1499. doi:10.1007/s11269-019-5431-x.

[6] Aili, T., A. Soncini, A. Bianchi, G. Diolaiuti, C. D’Agata, and D. Bocchiola. “Assessing Water Resources Under Climate Change in High-Altitude Catchments: a Methodology and an Application in the Italian Alps.” Theoretical and Applied Climatology 135, no. 1–2 (January 11, 2018): 135–156. doi:10.1007/s00704-017-2366-4.
[7] Zhang, Yiqing, Yi Luo, Lin Sun, Shiyin Liu, Xi Chen, and Xiaolei Wang. “Using Glacier Area Ratio to Quantify Effects of Melt Water on Runoff.” Journal of Hydrology 538 (July 2016): 269–277. doi:10.1016/j.jhydrol.2016.04.026.

[8] Treichler, Désirée, Andreas Kääb, Nadine Salzmann, and Chong-Yu Xu. “Recent Glacier and Lake Changes in High Mountain Asia and Their Relation to Precipitation Changes.” The Cryosphere 13, no. 11 (November 13, 2019): 2977–3005. doi:10.5194/tc-13-2977-2019.

[9] Momb lanch, Andrea, Lamp rini Papadimitriou, Sanjay K. Jain, Anil Kulkarni, Chandra S.P. Ojha, Adelayo J. Adeloye, and Ian P. Holman. “Untangling the Water-Food-Energy-Environment Nexus for Global Change Adaptation in a Complex Himalayan Water Resource System.” Science of The Total Environment 655 (March 2019): 35–47. doi:10.1016/j.scitotenv.2018.11.045.

[10] Liu, Y.R., Y.P. Li, Y. Ma, Q.M. Jia, and Y.Y. Su. “Development of a Bayesian-Copula-Based Frequency Analysis Method for Hydrological Risk Assessment – The Naryn River in Central Asia.” Journal of Hydrology 580 (January 2020): 124349. doi:10.1016/j.jhydrol.2019.124349.

[11] Chao, Lijun, Ke Zhang, Zhijia Li, Jingfeng Wang, Cheng Yao, and Qiaoling Li. “Applicability Assessment of the CASCade Two Dimensional SEDiment (CASC2D - SED) Distributed Hydrological Model for Flood Forecasting Across Four Typical Medium and Small Watersheds in China.” Journal of Flood Risk Management 12, no. S1 (January 28, 2019). doi:10.1111/jfr3.12518.

[12] Farhan, Suhaib Bin, Yinsheng Zhang, Adnan Aziz, Haifeng Gao, Yingzhao Ma, Jamil Kazmi, Atif Shahzad, et al. “Assessing the Impacts of Climate Change on the High Altitude Snow- and Glacier-Fed Hydrological Regimes of Astore and Hunza, the Sub-Catchments of Upper Indus Basin.” Journal of Water and Climate Change 11, no. 2 (November 7, 2018): 479–490. doi:10.2166/wcc.2018.107.

[13] Hayat, Huma, Adnan Ahmad Tahir, Sara Wajid, Arshad Mehmoond Abbassi, Fatima Zubair, Zia ur Rehman Hashmi, Asif Khan, Asim Jahangir Khan, and Muhammad Irshad. “Simulation of the Meltwater Under Different Climate Change Scenarios in a Poorly Gauged Snow and Glacier-Fed Chitral River Catchment (Hindukush Region).” Geocarto International (December 12, 2019): 1–17. doi:10.1080/10106049.2019.1700557.

[14] Ali, Syeda Saleha Fatim, Syed Amer Mehmood, Mujtaba Hassan, and Adil Latif. “River Runoff Modelling Through Geospatial Techniques-A Case Study of Snow and Glacier Fed Astore River Basin, Northern Pakistan.” 2019 Sixth International Conference on Aerospace Science and Engineering (ICASE) (November 2019): 1-12. doi:10.1109/icase48783.2019.9059146.

[15] Tahir, Adnan Ahmad, Samreen Abdul Hak eem, Tiesong Hu, Huma Hayat, and Muhammad Yasir. “Simulation of Snowmelt-Runoff Under Climate Change Scenarios in a Data-Scarce Mountain Environment.” International Journal of Digital Earth 12, no. 8 (September 2017): 910–930. doi:10.1080/17538947.2017.1371254.

[16] Meng, Fanhao, Chula Sa, Tie Liu, Min Luo, Jiao Liu, and Lin Tian. “Improved Model Parameter Transferability Method for Hydrological Simulation with SWAT in Ungauged Mountainous Catchments.” Sustainability 12, no. 9 (April 27, 2020): 3551. doi:10.3390/su12093551.

[17] Radecki-Pawlik, Artur, Andrzej Wałęga, Dariusz Młynski, Wojciech Młoeck, Rafał Kokoșzka, Tamara Tokarczuk, and Wiwiana Szalińska. “Seasonality of Mean Flows as a Potential Tool for the Assessment of Ecological Processes: Mountain Rivers, Polish Carpathians.” Science of the Total Environment 716 (May 2020): 136988. doi:10.1016/j.scitotenv.2020.136988.

[18] Boral, Soumita, Indra S. Sen, Dibakar Ghosal, Bernhard Peucker-Ehrenbrink, and Jordon D. Hemingway. “Stable Water Isotope Modeling Reveals Spatio-Temporal Variability of Glacier Meltwater Contributions to Ganges River Headwaters.” Journal of Hydrology 577 (October 2019): 123983. doi:10.1016/j.jhydrol.2019.123983.

[19] Han, Pengfei, Di Long, Zhongying Han, Mingda Du, Liyun Dai, and Xiaohua Hao. “Improved understanding of snowmelt runoff from the headwaters of China’s Yangtze River using remotely sensed snow products and hydrological modeling.” Remote Sensing of Environment 224 (2019): 44–59. doi:10.1016/j.rse.2019.01.041.

[20] YoosefDoost, Arash, Hossein Asghari, Reza Abunuri, and Mohammad Sadegh Sadeghian. “Comparison of CGCM3, CSIRO MK3 and HADCM3 Models in Estimating the Effects of Climate Change on Temperature and Precipitation in Taleghan Basin.” American Journal of Environmental Protection 6, no. 1 (March 5, 2018): 28–34. doi:10.12691/ajep-6-1-5.

[21] Valdes, Paul J., Edward Armstrong, Marcus P. S. Badger, Catherine D. Bradshaw, Fran Bragg, Michel Crucifix, Taraka Davies-Barnard, et al. “The BRIDGE HadCM3 Family of Climate Models: HadCM3@Bristol V1.0.” Geoscientific Model Development 10, no. 10 (October 12, 2017): 3715–3743. doi:10.5194/gmd-10-3715-2017.

[22] Li, Haojie, Hongyi Li, and Jian Wang. “Area Change of Snow and Ice in the Babao River Basin, Tibetan Plateau.” IGARSS 2019 - 2019 IEEE International Geoscience and Remote Sensing Symposium (July 2019): 4084-4087. doi:10.1109/igars.2019.8898368.
[23] Salman, Saleem A., Shamsuddin Shahid, Tarmizi Ismail, Kamal Ahmed, and Xiao-Jun Wang. “Selection of Climate Models for Projection of Spatiotemporal Changes in Temperature of Iraq with Uncertainties.” Atmospheric Research 213 (November 2018): 509–522. doi:10.1016/j.atmosres.2018.07.008.

[24] Javadinejad, Safieh, Kaveh Ostad-Ali-Araski, and Saeid Esllamian. “Application of Multi-Index Decision Analysis to Management Scenarios Considering Climate Change Prediction in the Zayandeh Rud River Basin.” Water Conservation Science and Engineering 4, no. 1 (March 2019): 53–70. doi:10.1007/s41101-019-00068-3.

[25] Yang, Qian, Shengbo Chen, Hongjie Xie, Xiaohua Hao, and Wenchun Zhang. "Application of snowmelt runoff model (SRM) in upper Songhuajiang Basin using MODIS remote sensing data.” In 2016 IEEE International Geoscience and Remote Sensing Symposium (IGARSS), pp. 4905–4908. IEEE, 2016. DOI: 10.1109/IGARSS.2016.7730280

[26] Maraun, Douglas. “Bias Correcting Climate Change Simulations - a Critical Review.” Current Climate Change Reports 2, no. 4 (October 10, 2016): 211–220. doi:10.1007/s40661-016-0050-x.

[27] Feicicabrino, James M., and Laurie D. Grigg. “A New GIS Landscape Classification Method for Rain/snow Temperature Thresholds in Surface Based Models.” Hydrology Research 48, no. 4 (October 24, 2016): 902–914. doi:10.2166/hr.2016.055.

[28] Ding, Lili, Ying Yang, Wei Wang, and Adrian Caneamir Calin. “Regional Carbon Emission Efficiency and Its Dynamic Evolution in China: A Novel Cross Efficiency-Malimqu Productivity Index.” Journal of Cleaner Production 241 (December 2019): 118260. doi:10.1016/j.jclepro.2019.118260.

[29] Alder, Jay R., and Steven W. Hostetler. “The Dependence of Hydroclimate Projections in Snow - Dominated Regions of the Western United States on the Choice of Statistically Downscaled Climate Data.” Water Resources Research 55, no. 3 (March 2019): 2279 – 2300. doi:10.1029/2018WR023458.

[30] Pangali Sharma, Til Prasad, Jiahua Zhang, Narendra Raj Kahanal, Foyez Ahmed Prodhawn, Basanta Paudel, Lamei Shi, and Nirdesh Nepal. “Assimilation of Snowmelt Runoff Model (SRM) Using Satellite Remote Sensing Data in Budhi Gandaki River Basin, Nepal.” Remote Sensing 12, no. 12 (June 17, 2020): 1951. doi:10.3390/rs12121951.

[31] Hao, Gai-rui, Jia-ke Li, Kang-bin Li, Kang Huang, Jia-bao Song, and Huai-en Li. “Improvement and Application Research of the SRM in Alpine Regions.” Environmental Science and Pollution Research 26, no. 36 (November 19, 2019): 36798–36811. doi:10.1007/s11356-019-06814-3.

[32] Kraaijenbrink, P. D. A., M. F. P. Bierkens, A. F. Lutz, and W. W. Immerzeel. “Impact of a Global Temperature Rise of 1.5 Degrees Celsius on Asia’s Glaciers.” Nature 549, no. 7671 (September 2017): 257–260. doi:10.1038/nature23878.

[33] Saha, Sourav, Lewis A. Owen, Elizabeth N. Orr, and Marc W. Cafer. “High-Frequency Holocene Glacier Fluctuations in the Himalayan-Tibetan Orogen.” Quaternary Science Reviews 220 (September 2019): 372–400. doi:10.1016/j.quascirev.2019.07.021.

[34] Coppola, Erika, Francesca Raaffaele, and Filippo Giorgi. “Impact of Climate Change on Snow Melt Driven Runoff Timing over the Alpine Region.” Climate Dynamics 51, no. 3 (August 31, 2016): 1259–1273. doi:10.1007/s00382-016-3331-0.

[35] Walton, Daniel B., Alex Hall, Neil Berg, Marla Schwartz, and Fengpeng Sun. “Incorporating Snow Albedo Feedback into Downscaled Temperature and Snow Cover Projections for California’s Sierra Nevada.” Journal of Climate 30, no. 4 (February 7, 2017): 1417–1436. doi:10.1175/jcli-d-16-0168.1.

[36] Coles, A.E., B.G. McDonkey, and J.J. McDonnell. “Climate Change Impacts on Hillslope Runoff on the Northern Great Plains, 1962–2013.” Journal of Hydrology 550 (July 2017): 538–548. doi:10.1016/j.jhydrol.2017.05.023.

[37] Bosshard, Thomas, Sven Kotlarski, Massimiliano Zappa, and Christoph Schär. “Hydrological Climate-Impact Projections for the Rhine River: GCM–RCM Uncertainty and Separate Temperature and Precipitation Effects*.” Journal of Hydrometeorology 15, no. 2 (April 1, 2014): 697–713. doi:10.1175/jhm-d-12-098.1.

[38] Pangali Sharma, Til Prasad, Jiahua Zhang, Narendra Raj Kahanal, Foyez Ahmed Prodhawn, Basanta Paudel, Lamei Shi, and Nirdesh Nepal. “Assimilation of Snowmelt Runoff Model (SRM) Using Satellite Remote Sensing Data in Budhi Gandaki River Basin, Nepal.” Remote Sensing 12, no. 12 (June 17, 2020): 1951. doi:10.3390/rs12121951.

[39] Jin, Haoyu, Qin Ju, Zhongbo Yu, Jie Hao, Huanghe Gu, Hanen Gu, and Wei Li. “Simulation of Snowmelt Runoff and Sensitivity Analysis in the Nyang River Basin, Southeastern Qinghai-Tibetan Plateau, China.” Natural Hazards 99, no. 2 (September 5, 2019): 931–950. doi:10.1007/s11069-019-03784-0.

[40] Tahir, Adnan Ahmad, Samreen Abdul Hakeem, Tiesong Hu, Huma Hayat, and Muhammad Yasir. “Simulation of Snowmelt-Runoff under Climate Change Scenarios in a Data-Scarce Mountain Environment.” International Journal of Digital Earth 12, no. 8 (September 2017): 910–930. doi:10.1080/17538947.2017.1371254.
Assessment Resistance Potential to Moisture Damage and Rutting for HMA Mixtures Reinforced by Steel Fibers

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Abstract

Rutting is mainly referring to pavement permanent deformation, it is a major problem for flexible pavement and it is a complicated process and highly observed along with many segments of asphalt pavement in Iraq. The occurrence of this defect is related to several variables such as elevated temperatures and high wheel loads. Studying effective methods to reduce rutting distress is of great significance for providing a safe and along-life road. The asphalt mixture used to be modified by adding different types of additives. The addition of additives typically excesses stiffness, improves temperature susceptibility, and reduces moisture sensitivity. For this work, steel fibres have been used for modifying asphalt mixture as they incorporated in the specimens by three percentages designated as 0.5, 1.0 and 1.5% by the weight of asphalt mixture. The evaluation process based on conducting Marshall Test, Compressive strength test, and the wheel tracking test. The optimum asphalt content was determined for asphalt mixture. The results of the Marshall quotient and the index of retained strength of modified mixtures were increased by 44.0 and 17.38% respectively with adding 1.0% of steel fibres compared with the conventional mixture. The rut depth and dynamic stability were determined by using a wheel tracking test at two various testing temperatures of 45 and 55°C and two applied stresses of 70 and 80 psi. Results show that adding 1% of steel fibres to asphalt mixtures is very effective in increase the rutting resistance and reduce moisture damage.

Keywords: Rutting Resistance; Moisture Damage; Dynamic Stability; Steel Fibers; Wheel Tracking Test.

1. Introduction

Rutting was a major problem for flexible paving. It is a longitudinal depression in the wheel paths of the roadways, Rutting is the most common is pavement permanent deformations due to repetitive loads of traffic that accumulation of small permanent strains of pavement materials after passing each axle load after a certain time. The design of the asphalt pavement is aimed to calculate the thickness of the pavement layers required to carry repeated loads safely under environmental conditions without significant deformation depend on the properties of the materials used in asphalt pavement layers [1]. The most important properties of the materials used in flexible pavement construction are a cost-effective influence to project specifications. Many factors should be considered to quality control procedures such as long economic life, environmental sustainability, low construction and repair time, low maintenance costs, use of waste materials [1, 2].

The asphalt pavement may be modified by adding various types of additives. The addition of additives normally improves the stiffness and increase temperature susceptibility. The increase of stiffness leads to improve the rutting
Fibers have provided high modulus, durability, resistance, and deformation resistance for asphalt pavement, and hence with more tensile resistance [5]. Also, fibers have been used as reinforcement of polymer mixtures to be used in other manufactures [6, 7]. These fibers provide the composites with strength and stiffness, therefore, allowing the mixture to better transfer the loads between fibers. Also, the addition of fibers in asphalt mix improves the mixture properties and contributes to sustainability by reducing road maintenance and extending its service life. Using fibers improves the mechanical performance of asphalt mixture, and fibers reinforce hot mix asphalt through a tridimensional network and improving the adhesion in the mix [8].

Mohammed et al. (2020) showed that adding steel fibers increased asphalt mixtures stiffness and presented an improvement in low temperature cracking resistance. Modifying asphalt mixture by 2% of steel fibers leads to improve fatigue life and a higher increase in indirect tensile strength [9]. Kureshi et al. (2019) modified asphalt concrete properties by using steel fibers, the surface thickness can be decreased by up to 25 or 30% with the improve in pavement performance, so, reducing total pavement maintenance costs. That observed the long-ridged steel fiber better results in the parameter chosen and an addition of 0.3% of fibers content improves the performance of dense Bituminous mixtures [10].

Çetin (2014) showed that using Low Carbon Structure Fibers (LCSF) to reinforce asphalt hot mixtures showed acceptable performance in both wearing and binder course [11]. Guo (2014) evaluated the dynamic stability of the asphalt concrete modified by steel fibers at various temperatures, the test uses 1, 2, 3 and 4 % of steel fibers mixing, concludes that adding 2 % of steel fibers to asphalt concrete will significantly improve asphalt pavement performance [12].

Aniruddh and Berwal (2016) studied the efficiency of asphalt concrete Mixtures modified with steel fibers in different quantities (2, 2.5, 3, 3.5, 4, 4.5, 5, 5.5 and 6%) with a length of 18 mm and 11 mm were tested during the conduct of the Marshall stability test for control and modified mixtures. The results presented that a higher improvement in the stability of asphalt mixture was at the optimum percent of added steel fibers for asphalt mixture at 3.5% of 11 mm length steel fiber. so, this fibers content was suggested for the improvement of the bituminous mix parameters [13].

Köfteci (2018) used different amounts of Iron Wire fibers (1%, 3%, 5%, 7%, and 9%) to reach the optimum fibers amount, the results of the investigations show that adding low-cost iron fibers by 3% improve the efficiency of asphalt mixtures [14]. Al-Kaissi et al. (2017) noticed that adding steel fibers to asphalt mixture improved the rutting resistance and modified the asphalt mixture with 0.2 % of steel fibers increased the dynamic stability about 6.4 % compared with the conventional mixture [15].

This research aims to evaluate the efficiency of using steel fibers for enhancement of asphalt mixtures prepared with locally available materials to resist rut occurrence of wearing course in flexible pavement depending on two temperatures and two stresses compared with conventional asphalt mixture and determined the moisture damage by finding the index of retained strength.

2. Materials and Methods

The locally available materials were selected to reach this work and that is economically beneficial. Asphalt cement AC (40-50), aggregate (coarse, fine), and filler were used. The steel fibers were used in this work as an additive, four asphalt concrete mixtures have been prepared in this work for wearing course layer (the conventional mixture and the asphalt mixtures modified by 0.5, 1 and 1.5 % of steel fibers).

The research methods were involved three stages, the first stage included Marshall test to find Optimum Asphalt Content and Marshall quotient, the second stage covered the evaluation of the compressive strength to calculate Index of Retained Strength (IRS), and the third stage included wheel tracking test to calculate the rutting depth and dynamic stability for asphalt mixtures.

2.1. Asphalt

AC (40-50) is the most commonly used in pavement construction in Iraq. It brought from Al-Daour refinery. The results of asphalt binder tests according to the SCRB (2003) [16]. Table 1 shows asphalt cement physical properties.
Table 1. Asphalt cement physical properties

| Test                                      | Units   | Results | SCRB 2003 Specification Limits | ASTM Specification No. |
|-------------------------------------------|---------|---------|-------------------------------|------------------------|
| Penetration, (25 °C, 100 g, 5 sec)       | 1/10 mm | 47      | 40 – 50                       | D-5                    |
| Ductility, (25 °C, 5 cm/min)             | cm      | 135     | ≥ 100                         | D-113                  |
| Kinematic Viscosity at 135 °C            | cSt     | 405     | –                             | D-2170                 |
| Softening point (Ring & Ball)            | °C      | 50      | –                             | D-36                   |
| Flash Point (Cleveland Open Cup)         | °C      | 270     | 232 min.                     | D-92                   |
| Specific gravity at 25 °C                |         | 1.04    | –                             | D-70                   |

**After Thin-Film Oven Test (ASTM D 1754)**

| Retained Penetration of Residue, (25 °C, 100 gm, 5 sec) | %  | 60 | 55 (Min) | D 5 |
| Ductility, (25 °C, 5 cm/min)                | cm | 82 | > 25     | D-113 |

2.2. Aggregates

Coarse crushed aggregate was used and obtained from Al-Nibaie quarry. Coarse aggregate sizes range for wearing course is between 12.5 mm and No.4 sieve (4.75 mm). Fine aggregate was bought from a local source (particle size between No.4 and No. 200). Laboratory evaluation described the basic properties of the aggregate. The results are presented in Table 2 according to the specification limit (SCRB, 2003) [16].

Table 2. Coarse and fine aggregate physical properties

| Property                                      | ASTM Specification No. | Result | SCRB Specification Limits |
|-----------------------------------------------|-------------------------|--------|---------------------------|
| **Coarse aggregate**                          |                         |        |                           |
| Bulk Specific Gravity                         | C-127                   | 2.579  | ---                       |
| Apparent Specific Gravity                     | C-127                   | 2.601  | ---                       |
| Percent of Water Absorption                   | C-127                   | 0.54   | ---                       |
| Percent of Wear (loss angels' abrasion)       | C-131                   | 15.79  | 30 Max                    |
| **Fine aggregate**                            |                         |        |                           |
| Bulk Specific Gravity                         | C-128                   | 2.61   | ---                       |
| Apparent Specific Gravity                     | C-128                   | 2.632  | ---                       |
| Percent of Water Absorption                   | C-128                   | 0.952  | ---                       |

2.3. Mineral Filler

Limestone dust was used for preparing asphalt concrete mixture in this work. The filler which passes through sieve opening (0.075 mm). Filler was bought from the lime factory, Karbala, Iraq. The mineral filler physical properties are shown in Table 3.

Table 3. Mineral filler physical properties

| Property                  | Result |
|---------------------------|--------|
| Passing No.200, (%)       | 95     |
| Specific gravity          | 2.71   |

2.4. Additives (Steel Fibers)

The steel fibers used were straight steel fibers bought from the Jingjiang Hongtu steel fiber factory, china. Each steel fiber is about 0.56 mm in diameter and about 30.1 mm length. Table 4 shows the physical properties, and Figure 1 shows the straight steel fibers used in this work.

Table 4. Steel fibers physical properties

| Description               | Straight |
|---------------------------|----------|
| Length, (mm)              | 30.1     |
| Diameter, (mm)            | 0.56     |
| Density, (Kg/m³)          | 720.6    |
| Tensile Strength Fu, (MPa)| 1185     |
| Aspect Ratio              | 54       |
| Price, ($/Ton)            | 1000     |
2.5. Selection of Aggregate and Filler Gradation

Aggregate and filler gradation were selected according to the specification of the (SCRB, 2003) [16], with a nominal maximum size 12.5 (mm), the gradation was selected (wearing course type IIIA). Selected aggregate gradation is showed in Table 5 and Figure 2.

Table 5. Selected gradation (12.5 mm nominal maximum size, wearing course type IIIA) (SCRB, 2003)

| English Sieve | Sieve opening, (mm) | Passing by Weight, (%) | Selected gradation, (%) | Specification range, (%) |
|---------------|---------------------|------------------------|-------------------------|--------------------------|
| 3/4"          | 19                  | 100                    | 100                     |                          |
| 1/2"          | 12.5                | 95                     | 90-100                  |                          |
| 3/8"          | 9.5                 | 83                     | 76-90                   |                          |
| No. 4         | 4.75                | 59                     | 44-74                   |                          |
| No. 8         | 2.36                | 43                     | 28-58                   |                          |
| No. 50        | 0.3                 | 13                     | 5-21                    |                          |
| No. 200       | 0.075               | 7                      | 4-10                    |                          |

Figure 2. Selected gradation and specification limits for wearing course (12.5 mm nominal maximum size) [16]
2.6. Marshall Test

Calculated the bulk specific gravity and density according to the procedure (ASTM D2726-08) [17]. Theoretical (maximum) specific gravity was conducted to the procedure (ASTM D2041-03) [17], and the percent of air voids according to the procedure (ASTM D3203-05) [17] for each specimen. Stability and flow values were calculated for all specimens according to the procedure (ASTM D6927-04) [17] and then calculate the Marshall quotient for all mixtures. The percent of air voids have been determined from equation 1 below:

\[
\text{Air Voids, (\%) } = \left(1 - \frac{\text{Bulk SG}}{\text{Max. theoretical SG}}\right) \times 100
\]  

(1)

2.7. Preparation of Conventional and Modified Mixture

The aggregate was sieved for every single size, then blended with filler to reach the desirable gradation for wearing course type IIIA according to the (SCRB, 2003) [16]. Heated the combined aggregate to temperature 155 °C at the same time asphalt was being heated to temperature 163 °C as the upper limit for (40-50) penetration grade. Then, asphalt binder was added with knowing quantity to the heated combined aggregate to reach the required quantity, finally, they mixed about (2 min) until the combined aggregate was coated with asphalt binder.

The preparation of modified mixture with steel fibers differs from a virgin mixture through the amount of additive steel fibers that are added by the total weight of mix as percentage 0.5, 1 and 1.5 % steel fibers that added to the asphalt mixture at temperature 155 °C and mixed about two minutes until the combined aggregate and steel fibers were coated with asphalt cement, Figure 4 presents a group of Marshall specimens.
2.8. Compressive Strength Test

The compressive strength test followed (ASTM D1074-17) [17] procedure. The compressive strength is the capacity of pavement materials to resist axially directed compressive forces. The compressive strength is one of the very important factors determining its suitability for use under the given load and environmental conditions as a highway paving material. The cylindrical specimens were (4.0 in.) in diameter and (4.0 in.) in height, were compacted using a compressive tool and the compaction process will stop when the specified height of the specimen was achieved to reach the target density and Figure 5 illustrated a group of specimens.

2.9. Index of Retained Strength Test

The test is conducted according to (ASTM D1075-11) [17]. This test covers the effect of water on compacted asphaltic mixtures, which is indicating for moisture susceptibility of mixtures. For this reason, prepared six specimens for each four mixtures and divided into two groups each group contains three specimens. Where group one was placed at a controlled 25 °C air bath for 4 hours and after that subjected to compressive strength. Group two placed at controlled 60 °C for 24 hours in a water bath and then placed at controlled 25 °C for 2 hours in another water bath and, then subjected to compressive strength as shown in Figure 6.
2.10. Wheel Tracking Test

A wheel tracking device used to predict rut depths and dynamic stability of the field for specific projects based on the rut depth of the laboratory wheel track. Simulation tests evaluating HMA quality have been used by rolling a small loaded wheel machine repeatedly through a prepared HMA sample, this test measures the rut depth and DS at a stress level of 70 psi and 80 psi applied to rectangular slabs of dimensions (300×200×50) mm at a different test temperature (45 °C and 55 °C) for 5000 cycles, as shown in Figure 7.

![Figure 7. Wheel tracking machine at university of Baghdad [18]](image)

2.11. Preparation Specimens Using Roller Compactor

The asphalt concrete mixture was heated to (150 °C) then papers were placed inside the mould after covering the internal surfaces of the mould with a layer of oil. At the same time mould of roller compactor was heated for casting the mixture in, the spatula was used for levelling the mixture and then put the mould in the device for compacting. There are two options for compacting the slab specimen either compacting to a target density or target height, but in this work, the target density (2.3 gm/cm³) was achieved. The slabs specimens were compacted and left cooling in the mould for (24) hours and then they extraction from moulds and cutting slabs for a wheel tracking test. Figure 8 presents roller-compacted used in this study and Figure 9 presents a group of wheel tracking slabs.

![Figure 8. Dyna-Compact roller compaction machine at NCCLR [19]](image)
3. Results and Discussion

3.1. Marshall Test

A group of Marshall tests (stability, density, and Air voids) was analyzed to find the OAC for asphalt concrete mixtures AC (40-50) and five different asphalt contents for each mixture range from 4 to 6 per cent (by weight of total mix) with an increase of 0.5 %. Three specimens were prepared and tested for each mixture by using aggregate (19 mm maximum size gradation). The OAC was (4.93 %) for conventional and modified mixtures. The Marshall properties for OAC were of the tests meet the Iraqi requirement specification of (SCRB, 2003) [16].

The results showed that the Marshall quotient was increased by 9.62, 44.0 and 19.815% compared with conventional mixture when using modified asphalt mixtures with 0.5%, 1%, and 1.5% steel fibers respectively with aggregate gradation of (SCRB, 2003) [16]. It was concluded that all the properties of the bituminous Concrete have been improved with the addition of the steel fibers. This is probably due to the well-distributed steel fibers in different directions of the bituminous matrix, which are highly resistant to shear displacement and strongly prevent any movement of aggregate particles as presented in Figure 10.

The maximum Marshall Quotient value was obtained at 1% fibers content. Then Marshall Quotient values decreased. The reason is due to the fibers may not distribute homogeneously in the mixture. Generally, applied loads are taken by the aggregate mass by means of contact points in Marshall samples. Because fibers created clustering in the mixture, as a result, the aggregate particles cannot be fully interlocked and contact between aggregate particles may be lost.
3.2. Compressive Strength Test

Evaluate moisture damage susceptibility by using Index of Retaining Strength (IRS) according to ASTM D-1074 and D-1075 [17]. The SCRB (2003) [16] pointed to the acceptable value of the IRS is (70%) or above which calculated as the ratio of compressive strength for conditioned specimens to the unconditioned specimens. The modified asphalt mixtures with steel fibers were having the highest effect in wet condition, where the wet compressive strength improved by 28.04, 71.96 and 52.03% for the same content of steel fibers, the compressive strength test results divulge that modified asphalt with Steel Fibers less sensitivity to moisture damage than conventional mixture. The results of the IRS at the OAC for modified mixtures with 0.5, 1 and 1.5% of steel fibers higher than the conventional mixture by 4.55, 17.38 and 10.76% respectively, and this lead to reduce the moisture sensitivity of asphalt pavement, and the moisture resistance improve with the improve of IRS, Figure 11 stated the Index of Retained Strength test results.

All modified mixtures show higher compressive strength than the control mixture. This may be due to the improvement in the stiffness of the asphalt mixture. The presence of additive may strengthen the bonding between the aggregates provided by the binder and thereby enhancing the aggregate to aggregate contact. This will result in increasing the resistance to crushing for asphalt pavement.

3.3. Wheel Tracking Test

The dynamic stability (DS) of the tested asphalt mixtures has been calculated according to Equation 2 [20]:

\[
DS = \frac{N_2 - N_1}{d_2 - d_1} \times C_1 \times C_2
\]

(2)

Where: DS is dynamic stability (cycle/mm), N_1 is 3750 cycle, N_2 is 5000 cycle, d_1 is rut depth corresponding to N_1 (mm), d_2 is rut depth corresponding to N_2 (mm), C_1 and C_2 are coefficients equal one in this study.

Figures 12 to 15 show that modified asphalt mixtures with steel fibers have higher dynamic stability compared with the conventional mixture, Figure 12 shows a reduction in rut depth was 31.15, 65.57 and 49.18% with the addition of 0.5, 1 and 1.5% steel fibers, respectively, at temperature 45 °C and stress 70 psi, into the conventional mixture, and Figure 13 present the reduction in rut depth was 17.14, 70.48 and 44.76% with the addition of 0.5, 1 and 1.5% steel fibers, respectively, at temperature 55 °C and stress 70 psi, into the conventional mixture, and Figure 14 shows the reduction in rut depth was 16.44, 60.27 and 34.25% with the addition of 0.5, 1 and 1.5% steel fibers, respectively, at temperature 45 °C and stress 80 psi, into the conventional mixture and Figure 15 present the reduction in rut depth was 19.53, 51.56 and 38.28% with the addition of 0.5, 1 and 1.5% steel fibers, respectively, at temperature 55 °C and stress 80 psi, into the conventional mixture. This is probably due to the well-distributed steel fibers in different directions of asphalt pavement.
the bituminous matrix, which are highly resistant to shear displacement and strongly prevent any movement of aggregate particles, therefore, the rutting resistance of the mixture increasing.

The dynamic stability values decreased and the rutting depth increased at the high content of steel fibers in asphalt mixture. The reason is due to the fibers may not distribute homogeneously in the mixture and created clustering in the mixture, as a result, the aggregate particles cannot be fully interlocked and contact between aggregate particles may be lost.

![Figure 12. Present the effect of steel fibers on dynamic stability and rutting depth for (5000 cycle, 45 °C, 70 psi)](image)

![Figure 13. Present the effect of steel fibers on dynamic stability and rutting depth for (5000 cycle, 55 °C, 70 psi)](image)
4. Conclusions

- Based on Marshall design method (stability and density voids analysis), the O.A.C was determined to be 4.93% for conventional and modified asphalt mixtures using AC (40-50). Furthermore, the magnitude of the Marshall quotient was increased by 9.62, 44.0 and 19.82% for mixtures modified by steel fibers with 0.5, 1.0 and 1.5% of respectively.
• The modified asphalt mixtures with 0.5, 1 and 1.5 % of steel fibers have an index of retained strength higher than conventional mixture by 4.55, 17.38 and 10.76% respectively. This referred to much more moisture damage resistance.
• The results of the wheel tracking test for modified asphalt mixtures with steel fibers showed higher values of dynamic stability compared with the conventional mixture.
• Reinforced asphalt mixtures with 0.5, 1.0 and 1.5 % of steel fibers have a rut depth lower than the conventional mixture by 31.15, 65.57 and 49.18 % respectively for 5000 cycles at 45 °C and 70 psi applied stress while this reduction values became 17.14, 70.48 and 44.76% as the temperature elevated to 55 °C. When the stress level increased to 80 psi, the same trend was observed.
• The overall results marked the magnitude of optimal steel fibers to be 1.0 %. Even though some additional cost will be witnessed, yet the gainful benefit from reducing the maintenance cost highly justify the worthily of using such fibers.

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7. Conflicts of Interest
The authors declare no conflict of interest.

8. References
[1] Agar E, Oztas G, Sutas L., “Concrete roads”, Istanbul, Istanbul Technical University Press, (1998).
[2] Arslan, M., S. Subasi, G. Durmus, O. Carter, and K. Stars. “Design and construction of concrete road concrete cladding alternative method research.” Ankara: Gazi University Scientific Research Projects (2007).
[3] Al-Shaybani, Mohammed Aziz Hameed. "Wheel Track Test to Predict Permanent deformation (Rutting depth) of Hot-Mix Asphalt Pavements and Using Silica Fume to Reduce Effect of Permanent Deformation." Journal of Kerbala University 16, no. 1 (2018): 104-113.
[4] Uomoto, Takeo, Hiroshi Mutsuyoshi, Futoshi Katsuki, and Sudhir Misra. “Use of fiber reinforced polymer composites as reinforcing material for concrete.” Journal of materials in civil engineering 14, no. 3 (2002): 191-209. doi:10.1061/(ASCE)0899-1561(2002)14:3(191).
[5] Salari, Zohreh, Behnam Vakhshouri, and Shami Nejadi. “Analytical Review of the Mix Design of Fiber Reinforced High Strength Self-Compacting Concrete.” Journal of Building Engineering 20 (November 2018): 264–276. doi:10.1016/j.jobe.2018.07.025.
[6] Elanchezhian, C., B.Vijaya Ramnath, G. Ramakrishnan, M. Rajendrakumar, V. Naveenkumar, and M.K. Saravanakumar. “Review on Mechanical Properties of Natural Fiber Composites.” Materials Today: Proceedings 5, no. 1 (2018): 1785–1790. doi:10.1016/j.matpr.2017.11.276.
[7] Sood, Mohit, and Gaurav Dwivedi. “Effect of Fiber Treatment on Flexural Properties of Natural Fiber Reinforced Composites: A Review.” Egyptian Journal of Petroleum 27, no. 4 (December 2018): 775–783. doi:10.1016/j.ejpe.2017.11.005.
[8] Sleti-Acevedo, Carlos J., Pedro Lastra-González, Pablo Pascual-Muñoz, and Daniel Castro-Fresno. “Mechanical Performance of Fibers in Hot Mix Asphalt: A Review.” Construction and Building Materials 200 (March 2019): 756–769. doi:10.1016/j.conbuildmat.2018.12.171.
[9] Mohammed, Monketh, Tony Parry, Nick Thom, and James Grenfell. “Microstructure and Mechanical Properties of Fibre Reinforced Asphalt Mixtures.” Construction and Building Materials 240 (April 2020): 117932. doi:10.1016/j.conbuildmat.2019.117932.
[10] Shahrukh Kureshi, Neha Duryodhan, Ashwini Jiwane, Rishi Dubey, Nidhi Vishwakarma, and KshmaDhanwate “To Study the Behavior of Asphalt Concrete Pavement Using Steel Wool Fiber”, International Research Journal of Engineering and Technology (IRJET) 6, no. 3 (2019): 3375-3381.
[11] Çetin, Sedat. “Evaluation on the Usability of Structure Steel Fiber-Reinforced Bituminous Hot Mixtures.” Construction and Building Materials 64 (August 2014): 414–420. doi:10.1016/j.conbuildmat.2014.04.093.
[12] Guo, Jie Fei. “The Effect of Steel Fiber on the Road Performance of Asphalt Concrete.” Applied Mechanics and Materials 584–586 (July 2014): 1342–1345. doi:10.4028/www.scientific.net/amm.584-586.1342.

[13] Aniruddh and Parveen Berwal, “An Experimental Study on Behaviour of Steel Fibre on Bituminous Mixes (Bitumen Concrete)”, International Journal of Current Engineering and Technology 6, no. 4 (2016): 1220-1223.

[14] Köfteci, Sevil. “Experimental Study on the Low-Cost Iron Wire Fiber Reinforced Asphalt Concrete.” Teknik Dergi (July 1, 2018): 8515-8535. doi:10.18400/tekderg.350135.

[15] Zainab Ahmed Al-Kaissi, Ahmed S. D. Al-Ridha and Sarah Muhammad Kareem “Improving the Rutting Resistance of Flexible Pavement Reinforced with Steel Fiber”, Imperial Journal of Interdisciplinary Research (IJIR) 3, no. 12 (2017): 2454-1362.

[16] SCRB, "Standard Specifications for Road and Bridge”. Section R/9, Hot-Mix Asphalt Concrete Pavement, Revised Edition, (2003), State Corporation of Roads and Bridges, Ministry of Housing and Construction, Republic of Iraq.

[17] ASTM, "Road and Paving Materials Vehicle Pavement Systems", Annual Book of ASTM Standards, (2018), Vol. 04 and Vol. 05, American Society for Testing and Materials.

[18] Abdulkhaleq Mahdi, Ali, and Mohammed Qadir Ismael. “Rutting Resistance Potential of High Modulus Asphalt Concrete Pavements.” Journal of Engineering and Applied Sciences 14, no. 12 (December 10, 2019): 4183–4190. doi:10.36478/jeasci.2019.4183.4190.

[19] Raof, Hussein Burhan, and Mohammed Qadir Ismael. “Effect of PolyPhosphoric Acid on Rutting Resistance of Asphalt Concrete Mixture.” Civil Engineering Journal 5, no. 9 (September 1, 2019): 1929–1940. doi:10.28991/cej-2019-03091383.

[20] Kyokai, N.D., “Pavement investigation and testing methods handbook [Hosou Chousa Shikenhou Binran]”, 3, Japan, Japan Road Association, (2007).
FEM Optimisation of Seepage Control System Used for Base Stability of Excavation

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Abstract

With the existence of a high groundwater level, the head difference between the inside and outside of an excavation may lead to the loss of stability of the excavation’s surface. Hence, a fundamental understanding of this occurrence is important for the design and construction of water-retaining structures. In some cases, the failure mechanism cannot be predicted exactly because of its mechanical complexity as well as a major lack of protection systems and not adopting effective countermeasures against this phenomenon. The article took a tranche from an 80 km long open sewer located in the Ruhr area, Germany as an example to establish a hydro-geological model and analyse the instability of the excavation base surface caused by the groundwater flow at 45m deep and to present the effectiveness of an adopted drainage system inside the excavation pit as 39 columns of sand to relax the pore water pressure. By using the Finite Element Method (FEM) analysis, the failure mechanism was investigated before applying any countermeasures, and the total length of the adopted countermeasure system was minimised. Also, various position tests were performed on the adopted drainage system to confirm the optimised position. The results of this numerical study allowed the deduction of the importance of the used drainage system by achieving 44% more in the excavating process. After achieving the required excavation depth, a further increase of the sand columns’ penetration may be considered non-economic because, after adding extra depth, all the situations have the same safety factor. In addition, this can provide a reference for the optimised position of the sand columns where they must be applied right by the wall and limited by a critical distance, D/2, half of the embedded depth of the wall.

Keywords: Factor of Safety; Failure Mechanism; Deep Circular Excavation; Finite Element Method; Drainage System.

1. Introduction

Deep excavation projects under retaining walls are increasing in recent years along with the growth of infrastructure needs and the development of urban sites for basements and tunnels. The limitation on open lands for further development is one of the reasons behind that trend. When the depth of a planned excavation pit is lower than the groundwater level in the field, the groundwater head difference between the inside and outside leads to basal failure of the excavation pit. In some cases, the failure mechanism cannot be predicted precisely because of its mechanical complexity, and it is extremely difficult to stop these phenomena once they start, as well as a notable lack of both protection systems and adoption of effective countermeasures against this phenomenon. In the design of deep excavation under conditions with a high groundwater level, several issues are related to the stability of the excavation. These are often dominated by the water flow around the wall and include upheaval failure, general shear failure and sand boiling failure [1].

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Therefore, for a clearer understanding of the basal failure phenomenon of the excavation pit and its mechanisms, various methods (theoretical, numerical and experimental) have been addressed in literature and reported in this work.

According to EN 1997-1/Eurocode 7 [2], heave, uplift, internal erosion or piping represents the failure mechanisms induced by seepage flow. Heave appears when the seepage forces developed by groundwater flow lift the soil on the downstream side. Also, when seepage forces are acting on the subsoil, especially the upward seepage close to the excavation level, where the seepage forces could be higher than the effective weight of the soil, failure by uplift occurs. Along with seepage, internal erosion or piping might occur with the transport of soil grains within a soil layer at the interaction surface of a soil layer and a structure by groundwater flow, or starting with a pipe-shaped canal on the downstream side and propagating backwards, respectively, which may finally lead to collapse, as shown in Figure 1.

The most used verification method was presented by Terzaghi (1943) [3]. He used model tests to study the influence of seepage flow on the stability of retaining excavations. He found that a rectangular prism, adjacent to the wall, could be assumed to be the failure shape lifted by pore water pressure. The total height of the rectangular prism corresponds to the embedment depth of the wall from the surface of the excavation pit to the toe of the excavation wall, and its width is half of the embedded depth of the wall. Experimental tests on the sand model were examined by Tanaka and Verruijt (1999) [4] to describe the mechanism of basal failure of the excavations. They found that the critical hydraulic head differences discovered by Terzaghi and prismatic failure are close to the head difference at the start of deformation. Based on the experimental results of twelve centrifugal model tests, Sun (2016) [5] presented two types of failure: general upheaval and local failure (sand piping and sand boiling). He compared the experimental results with calculated results obtained from the developed approach to the stability analysis of deep excavation against a confined aquifer. The results showed good agreement.

![Figure 1. The development of piping around waterfront structures according to EAU (2004)](image)

Considering a fixed wall, which may represent a sheet pile strut, Benmebarek et al. (2005) [6] used the explicit finite difference method FLAC 2D code to analyse the failure of sandy soil within a cofferdam subjected to an upward seepage flow. Their results identified different failure mechanisms at the downstream side occurring for critical hydraulic head loss, which were greatly influenced by the interface and soil characteristics. Benmebarek et al. (2014) [7] investigated the critical height of water on the upstream side of free and fixed sheet pile walls using the commercial code FLAC. They found that seepage failure by basal heaving given by Terzaghi’s discovery is less critical than the obtained rotational failure. Aulbach and Ziegler (2013) [8] numerically analysed the excavation stability of steady-state flow and presented a triangular prism of failure, its height and width being equal to the embedment depth of the wall and its half, respectively.

To study the base stabilities of retaining excavations against seepage failure by heave where a relatively permeable cohesionless soil layer lies above a less permeable soil layer between the excavation base and wall toe, Koltuk et al. (2019) [9] compared the results of numerical simulations and experimental tests with those obtained from Terzaghi and Peck’s approach. Both the numerical analysis and the model tests showed a good agreement in which a triangle-shaped heave zone with a larger width was obtained, unlike the rectangular-shaped heave zone suggested by Terzaghi and Peck.
The method of fragments was validated by Madanayaka and Sivakugan (2019) [10] to estimate the exit hydraulic gradient and flow rate of axisymmetric circular cofferdams for numerical and experimental models. The results that were determined for both exit hydraulic gradient and flow rate showed an excellent agreement between the numerical model results and experimental results, in which the numerical predictions were within ±5% of the corresponding experimental results. Koltuk and Azzam (2019) [11] performed numerical analyses and experimental tests of quicksand conditions in cohesionless soils to estimate the base stability of a retained excavation against seepage failure by heave. The results showed that the potential differences that led to failure were higher than the potential differences that were required for the theoretical approaches and the development of quicksand conditions was unachievable for the situation where a high permeable cohesionless soil layer was overlying a less permeable soil layer between the excavation base and wall tip.

Marsland (1953) [12] studied the mechanisms of failure using sand–water models. He found that the pit dimensions and soil conditions have a clear influence on the hydraulic failure of the excavation pit. For homogeneous soil conditions, the stability increases with increasing the embedment depth of the retaining wall below the excavation base and decreases as the excavation become narrower. Aulbach et al. (2013) [13] generated eight design charts, taking into consideration the geometrical boundary conditions, in order to determine the required embedded length of the excavation retaining system for safety against hydraulic heave. Their results showed clearly that the width has an enormous influence on safety. Faheem et al. (2003) [14] used the finite element method (FEM) to evaluate the two-dimensional base stability of excavations. They stated that the thickness of the soft soil layer between the excavation base and hard stratum, the ratio of the depth to the width of the excavations, the penetration depth of the wall below the excavation base and its stiffness all have a significant influence on the base stability of excavations. Zhang and Goh (2016) [15] used the two-dimensional FEM with shear strength reduction using the commercial software Plaxis 2011to analysis the basal heave stability of deep narrow braced excavations in terms of stability number and factor of safety. The results indicate that the stability number is independent of undrained shear strength and the system stiffness has a small influence on the factor of safety against basal heave.

A series of parametric studies were carried out by Pratama and Ou (2018) [16] using numerical analysis and the conventional method of flow net to identify the factor of safety values representing the safety against sand boiling and predict the failure mechanism caused by groundwater flow. The results show that the critical width of the soil prism presented by Terzaghi is bigger than that set by the author; likewise, the soil density and excavation geometry has some significant effects in governing the failure mechanism and the hydraulic gradient. To assess the coefficient of security against heaving and investigate the critical heads for the failure of hydraulic structures, Fontana (2008) [17] analysed different thickness sheet piles embedded in sandy soils. Theoretical values and experimental data were compared, showing good agreement if the ground surface deformation and the thickness of the sheet pile are properly taken into account.

Based on the experimental results of Yousefi et al. (2016) [18], where they simulated the seepage flow and its behaviour downstream of a sheet pile under inclined and vertical configurations using laboratory models to study the boiling phenomenon, an inclined configuration with an angle of 60° to the horizontal direction and a ratio of sheet pile depths d/D=0.34 and d/D=0.44, in which d is the vertical penetration depth of the sheet pile and D is the thickness of the foundation material for inclined and vertical configurations respectively, have successfully reduced the boiling phenomenon and exit vertical hydraulic gradient. Zhao et al. (2020) [19] conducted both an FEM and full-scale in-situ tests in Guangzhou, China, to study the effects of circular shaft diameters on failure by uplift and to analyse its failure mechanisms, as well as to determine a reasonable stability judgement method for circular shafts subjected to hydraulic uplift. The authors found that the observed phenomena on-site have good agreement with the results obtained from the finite element analysis. Pane et al. (2017) [20] presented a methodology based on the simple concept of increasing the drainage capacity of the embedded portion of the retaining walls aimed at minimising the risk of uplift and heave failures of retained excavations. They found that the draining wall can greatly improve heave safety, and the dominant failure mechanism changes from heave to uplift.

From all of the above, it can be seen that most of what has been done in literature did not take place in real excavations, and also focussed just on the analysis aspect without adopting effective countermeasures and providing new techniques for protecting excavation pits against basal failure. In this context, the present paper took a tranche from an 80 km long open sewer located in the Ruhr area, Germany (Figure 4) as an example to establish a hydro geological model and analyse the instability of the excavation base surface caused by the groundwater flow using the powerful geotechnical software, Plaxis v. (2012). The maximum achieved depth of the excavation and the failure mechanism before applying the drainage system was checked out and compared with previous research. As a next step, this study presented the affectivity of the adopted drainage system inside the excavation pit to relax the pore water pressure in order to achieve the required excavation depth. Finally, it optimised the length of the adopted drainage system and studied its position effect from the wall, taking into consideration the economic and safety aspects. The following flowchart (Figure 2) presents the methodology employed in this study.
2. Case Study

Since the period of industrialisation, wastewater in the Ruhrgebiet in Germany has drained into the east–west river of Emscher. Now, due to the world’s most modern sewage system, the river of Emscher will be converted into a near-natural body of water in order to restore the natural condition of the Emscher and its tributaries. The overall project is over a length of 51 km, between Dortmund-Deusen and the mouth of the Emscher in Dinslaken, where the wastewater will flow in closed piped channels of about 400 km of sewer tunnels with a maximum outside diameter of 4.20 metres that are up to 40 m deep.

In the area of section 40 (Figures 3 and 4), data from laboratory investigations have been collected where 300 boreholes were drilled to a depth of about 70 m in order to investigate the soil characteristics. The information gained mostly showed that the site consists of two main layers, Quaternary sand, predominantly with underlying cohesive soils such as marl. Figure 5 shows the schematics of the systematised geotechnical longitudinal section of construction section 40.
Figure 3. The main course of the Emscher

Figure 4. The site of the study area in section 40
Figure 5. The systematised, geotechnical longitudinal section of construction section 40

Figure 6 shows the grain size range of the Cretaceous ground (solid lines) passed through in the west half of section 40 and the grading distribution of the material of the Concordia-Sprung in the east half of section 40 (dashed lines).

Figure 6. Typical grading curves in the Cretaceous determined in the site investigation (solid lines) and typical grading curves in the Concordia-Sprung fault zone (dashed lines)

Various types of marl can be distinguished along the route of construction section 40 (Route of the Interceptor SD.033 - PW OB). The Emscher marl is the predominant deposit and consists of glauconitic, calcareous, clayey silts and calcareous silty fine sands, which are consolidated to clay or sand marl and clay or calcareous marlstone. Above the Emscher marl are the Osterfeld beds, which consist of marly and silty fine sands, mostly with a considerable medium sand content, as well as very sandy silts. The Bottrop beds, which partly overlie the Osterfeld beds, consist of grey-green glauconitic fine sand marlstone, which transitions at the base from marly fine to medium sands. The upper part of the Bottrop beds consists of marlstones and fine sandy clay marlstones. At its surface, the marl is mostly softened and weathered. The Cretaceous beds dip flatly to the north-northwest. Additional tectonic faults lead to all these Cretaceous strata being passed through by the tunnel drive.

In this paper, the western half of section 40 (PW OB) was taken as a case study, where the Emscher marl is the predominant deposit, falling under mostly a considerable medium sand.
3. Numerical Simulation of the Case Study

In both cases – cohesive soil and groundwater relaxing system – the classical method fails. Here, numerical simulations based on the FEM appear to be a helpful tool, since they present the relevant failure mechanism as a result of the calculation. The FE modelling method using the Plaxis 2D-V 2012 computer program has been applied to the case study of a deep excavation PW-OB located in section 40 (Figure 4). Using the numerical method advantages, an axisymmetric model is used where the FE mesh consists of 15-nodes of triangular elements. The size of the calculation model was chosen so that the boundaries do not influence the deformation behaviour of the model. Theoretically, the tensile stresses that can be absorbed by the ground are cut-off. In order to simulate the excavation and construction process as a real case, the calculation was divided into several steps based on the actual excavation planning. According to the data of the previous on-site investigation and the laboratory tests, the parameters, such as the permeability, the modulus and so on, for every soil are determined with the Mohr-Coulomb model used for all soil layers. The soil parameters for the simulation are summarised in Table 1.

Table 1. Main hydraulic and mechanical properties of the soils

| Parameter                     | Quaternary sand | Marl | Clean Sand |
|-------------------------------|-----------------|------|------------|
| Unsaturated unit weight γ_{unsat} (kN/m³) | 19              | 20   | 19         |
| Saturated unit weight γ_{sat} (kN/m³)     | 20              | 22   | 20         |
| Friction angle (°)            | 30              | 25   | 30         |
| Cohesion (kN/m²)              | 0               | 40   | 0          |
| Dilation angle (°)            | 0               | 0    | 0          |
| Poisson's ratio               | 0.3             | 0.25 | 0.33       |
| Wall-friction and -adhesion R | 0.5             | 0.2  | 1          |
| Permeability (m/s)            | 1x10⁻⁴          | 1x10⁻⁶| 1x10⁻³    |
| Young's modulus (kN/m²)       | 29,700          | 40,000| 20,200     |

Figure 7 presents the project where the case study was chosen for this paper. It shows a circular excavation with an inside diameter B=46 m and a depth d=45 m, as the three-dimensional model presented in Figure 8. The surrounding soil is retained by an impermeable wall of 2 m in thickness. The wall is inserted by D=6 m beneath the final excavation.

Figure 7. Presentation of the pit chosen for this paper (PWOB)

To relax the pore water pressure that causes excavation bottom instability due to the different groundwater level where the outside level is higher than inside the excavation, 39 clean sand columns with a diameter of 30 cm reaching down to a depth of te = 90 m from the ground surface, and 2m away from the wall were modelled as a concentric thin slot. Sandy columns with relatively high permeability (k=10⁻³ m/s) are an appropriate measure to improve the hydraulic situation at the bottom of the excavation. For the initial state, the groundwater was set at 6 m below the top of the site.
The mathematical modelling of the groundwater flow resulting from the excavation of the soil and simultaneous drainage via the retaining wall and the bottom of the construction pit is based on Darcy’s law. The effective stress was calculated in the form of a coupled analysis, i.e. the distribution of the pore water pressure determined in a calculation of flow and used as the initial condition for the subsequent stress calculation. The groundwater flow calculated in the respective excavation state and the calculated flow pressure describes the steady-state.

4. Results and Discussion

4.1. Overall Stability without Countermeasures

The following investigation deals with the verification against excavation bottom instability. The first part of this research work consists of evaluating the maximum excavation depth that can be reached without applying any countermeasures.

After calculating the initial stress state by initialising the stresses in the model with the coefficient $K_0$ of lateral pressure of the earth at rest $K_0 = 1 - \sin \phi$, the excavation states were displayed in 1 m excavation steps and here the calculation of the strain state under stress was coupled to the calculation of the groundwater flow.

The performed calculations indicate that the excavation process is safe enough without any countermeasures reaching down to a depth of 25 m from the ground surface. For a deeper excavation process, the situation would be exposed to the collapse of the excavation base. Here, the drainage system (sand columns) must be installed. In order to demonstrate the failure mechanism, the drilling process was attended up to 26 m, where the bottom of the excavation at that depth was affected by hydraulic failure.

Figure 9 shows the path of the groundwater within the soil of the studied case at the critical moment where the excavation base is exposed to collapse. The water located in the upper layer goes in a horizontal direction and accumulates at the wall front, then fast-flowing down creates an intensive upward seepage force at the downstream side. The reason for this is that the low permeability of the lower layer (marl) creates isolation at the soil interface leading to preventing water from passing down on the ground.

At the moment where the situation exposed to the collapse of the excavation base, Figure 10 shows the mechanism of failure. The soil at the base of the excavation lifted completely owing to the intensive upward seepage forces resulting from the ground stratification and their different soil permeability. It appears as general heave, and the prism of failure does not give a specific region.
Comparing the obtained failure mechanism from this study to those accomplished by Benmebarek et al. (2005 and 2014), Aulbach and Ziegler (2013) and Koltuk et al. (2019) [6-9] in which three types of failure have been presented (triangular, rectangular prisms and boiling) and the rectangular failure body with width D/2 represented by Terzaghi (1943) [3], shows clearly that they are not in good agreement. Therefore, it becomes clear that for real cases, where horizontal stratification exists between the excavation base and the wall tip and for specific soil characteristics, the mechanism of failure cannot be generalised to all situations, and its diagnosis varies from case to case.
4.2. Overall Stability with the Implemented Countermeasures

As countermeasures, the second part of this research is related to applying the drainage system and testing its effectivity against the failure of the excavation base due to the water seepage forces. For this project, the implemented drainage system consists of 39 columns of sand with a high coefficient of permeability reaching down 90 m from the ground surface. As the material of the drainage system consists of sand, the 39 clean sand columns can easily be excavated with the surrounding soil.

The results indicate that, for all the excavation states, the situations have the required safety to achieve the targeted depth of 45 m deep and the drainage system is very effective to absorb the seepage forces. However, it is not clear which failure mechanism becomes relevant in the case of excavations with a drainage system in the subsoil. It does not seem to be admissible to transfer the classical failure mechanisms to these situations.

In aiming to give the present case study more precise design values in economic terms, and taking into consideration safety as the first criterion, varied depths and positions of the drainage system have been analysed.

4.3. Optimisation of the Penetration Depth

In order to study the effect of the drainage system penetration beneath the subsoil, the length of the columns, \( t_e \), is reduced from the designed value, 90 m, by a step of 3.5 m, while keeping the same characteristics of the soils, until the occurrence of collapse at the base of the excavation. The results presented in Figure 11 indicate that for a penetration depth of 76 m of the drainage system beneath the subsoil, the geo-hydraulic situation is safe enough against the failure of the excavation base. At a further reduction in the penetration depth of the drainage system (less than 76 m), the situation falls, and collapse occurs. From a depth of penetration of 76 m to 83 m the safety factor (FS) went up slightly, taking the value from 1.1624 to 1.1647; however, it remained stable from 86.5 m to 90 m. That slight increase of FS value may be considered non-economic because, at a depth of penetration of 76 m, all the excavation states have the required safety. The reason for this could be that the flow path from upstream was limited by global driving contours, whereas the point of intersection for the lower limit of this with the drainage system was located at -76 m from the ground’s surface. The other deepest flow path can be considered non-influential on the behaviour of the phenomenon.

Also, the permeability of the drainage material has been reduced in order to test its effect on the stability of the excavation base. The results indicate that for permeability of less than \( k < 10^{-3} \text{ (m/s)} \), the seepage problem cannot be resolved no matter how deep the drainage system is.

4.4. Optimisation of the Position

In this part, the drainage system moved from the wall going to the centre of the pit with a distance \( d_e \) by a step of 0.5 m. The first position was attached to the wall with a distance of 0.25 m, as it is the nearest possible position to the wall. In each step, the factor of safety has to be gained for each position ending at the optimal position.
Figure 12 shows that from position D/2 of the drainage system closer to the wall, the factor of safety is raised and achieves the maximum value near the wall. In other words, from the other side with the position D/2 of the drainage system heading towards the middle of the pit, the process of the excavations fails before achieving the required depth (45 m deep) and the drainage system cannot solve the situation even if reaches very deep. Here, it could be noted that, for the analysis of the basal heave of excavations, the upward seepage flow from the upstream side is limited by the diving contours with a distance D/2 from the wall.

Figure 12. Effect of drainage system position

Comparing what was mentioned in the literature by Terzaghi (1943) [3], where the relevant zone suggested for seepage failure is a rectangular prism adjacent to the wall with D/2 in width (Figure 13), with the acceptable limit of drain positions developed from this study, they are obviously in good agreement. Lastly, through the obtained results, it could be supposed that for the stability of the excavation base against seepage flow, the mechanism of failure is not related to the vulnerable region.

Figure 13. The acceptable positioning of the drainage system
5. Conclusions

Safety against water flow in deep excavations represents a crucial aspect of design. In many cases, both the design and the overall cost of the excavation system are dictated by this problem of hydraulic failure. In this research, a real project of deep-braced excavation located in the Ruhrgebiet, Germany, subjected to seepage flow, was established using the elastic–plastic FEM to predict the failure mechanism caused by groundwater flow and to perceive the factor of safety values against the failure of the excavation base. A drainage system, consisting of clean sand with high permeability, was adopted and implemented for this case study to relax the excess pore water pressure. This has been numerically tested for its effectivity. To underscore the scientific value of this research work, the optimised length of the drainage system and its effective position from the wall have been analysed with regard to the economic aspect, bearing in mind safety as the first criterion. The results that have been mentioned in the literature were compared with those obtained by numerical simulations in this work. The conclusions are as follows:

- Before applying any countermeasures, the soil at the base of the excavation lifted completely owing to the intensive upward seepage forces resulting from the ground stratification and their different soil permeability. In that situation, the achieved excavation depth was slightly more than half of the required depth. The mechanism of failure appears as general heave, and the prism of failure does not give a specific region. Therefore, it becomes clear that for real cases, where horizontal stratification exists between the excavation base and the wall tip and for specific soil characteristics, the mechanisms of failure cannot be generalised to all situations, and their diagnosis varies from case to case.

- To achieve the required excavation depth, the adopted drainage system has been implemented. Results showed that the drainage system was quite an effective countermeasure against the failure of the excavation base.

- By analysing the effect of the drainage system penetration beneath the subsoil, results indicated that with 76 m in penetration depth, the drainage system could sufficiently support the geo-hydraulic situation against the failure of the excavation base. A slight increase in the safety factor when the drainage system reached down 90 m, however, may be considered a non-economic decision.

- From the position D/2 of the drainage system going closer to the wall, all excavation processes were safe enough until reaching the required depth and the factor of safety was raised and achieved the maximum value near the wall. Otherwise, the drainage system could not resolve the situation even if they reached very deep, and the process of the excavation failed before achieving the required depth.

- For similar projects subjected to hydraulic heave, the obtained results can be provided as a reference to use for stability evaluation with regards to the applicability of the adopted system and its efficacy of safety and economy.

5.1. Future Aspects of the Research

The overall performance of the drainage system, which consists of clean sand, to be reported as satisfactory. More research must be carried out in order to determine the possible effective countermeasures against a basal failure of excavation. The authors suggest treating the problems using the technique of stone columns with the installation effect for which it has the best correlation regarding the drainage and improvement of its surrounding soil.

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7. Conflicts of Interest

The authors declare no conflict of interest.

8. References

[1] Ou, Chang-Yu. “Deep Excavation” (April 21, 2014). doi:10.1201/9781482288469.

[2] “Eurocode 7. Geotechnical Design” (2004). doi:10.3403/03181153u.

[3] Terzaghi, Karl. “Theoretical Soil Mechanics. Johnwiley & Sons.” New York (1943): 11–15.

[4] Tanaka, Tsutomu, and Arnold Verruijt. “Seepage Failure of Sand behind Sheet Piles—The Mechanism and Practical Approach to Analyse —.” Soils and Foundations 39, no. 3 (June 1999): 27–35. doi:10.3208/sandf.39.3_27.
[5] Sun, Yu-yong. “Experimental and Theoretical Investigation on the Stability of Deep Excavations against Confined Aquifers in Shanghai, China.” KSCE Journal of Civil Engineering 20, no. 7 (January 22, 2016): 2746–2754. doi:10.1007/s12205-016-0488-3.

[6] Benmebarek, N., S. Benmebarek, and R. Kastner. “Numerical Studies of Seepage Failure of Sand within a Cofferdam.” Computers and Geotechnics 32, no. 4 (June 2005): 264–273. doi:10.1016/j.compgeo.2005.03.001.

[7] Benmebarek, N., A. Bensmaine, S. Benmebarek, and L. Belounar. “Critical Hydraulic Head Loss Inducing Failure of a Cofferdam Embedded in Horizontal Sandy Ground.” Soil Mechanics and Foundation Engineering 51, no. 4 (September 2014): 173–180. doi:10.1007/s11204-014-9274-8.

[8] Aulbach, Benjamin, and Martin Ziegler. “Simplified Design of Excavation Support and Shafts for Safety against Hydraulic Heave / Einfache Bemessung von Baugruben Und Schächten Im Hinblick Auf Die Sicherheit Gegen Hydraulischen Grundbruch.” Geomechanics and Tunnelling 6, no. 4 (August 2013): 362–374. doi:10.1002/geot.201300031.

[9] Koltuk, Serdar, Jie Song, Recep Iyisan, and Rafig Azzam. “Seepage Failure by Heave in Sheeted Excavation Pits Constructed in Stratified Cohesionless Soils.” Frontiers of Structural and Civil Engineering 13, no. 6 (September 27, 2019): 1415–1431. doi:10.1007/s11709-019-0565-z.

[10] Madanayaka, Thushara Asela, and Nagaratnam Sivakugan. “Validity of the Method of Fragments for Seepage Analysis in Circular Cofferdams.” Geotechnical and Geological Engineering 38, no. 2 (October 31, 2019): 1547–1565. doi:10.1007/s10706-019-0111-9.

[11] Koltuk, Serdar, and Rafig Azzam. “Use of Quicksand Condition to Assess the Base Stabilities of Sheeted Excavation Pits Against Seepage Failure in Cohesionless Soils.” Arabian Journal for Science and Engineering 44, no. 10 (May 2, 2019): 8515–8526. doi:10.1007/s13369-019-03890-y.

[12] Marsland, Arthur. “Model Experiments to Study the Influence of Seepage on the Stability of a Sheeted Excavation in Sand.” Géotechnique 3, no. 6 (June 1953): 223–241. doi:10.1680/geot.1953.3.6.223.

[13] Aulbach, Benjamin, Martin Ziegler, and Holger Schüttrumpf. “Design Aid for the Verification of Resistance to Failure by Hydraulic Heave.” Procedia Engineering 57 (2013): 113–119. doi:10.1016/j.proeng.2013.04.017.

[14] Faheem, Hamdy, Fei Cai, Keizo Ugai, and Toshiyuki Hagiwara. “Two-Dimensional Base Stability of Excavations in Soft Soils Using FEM.” Computers and Geotechnics 30, no. 2 (March 2003): 141–163. doi:10.1016/s0266-352x(02)00061-7.

[15] Zhang, Fan, and Anthony T. C. Goh. “Finite Element Analysis of Basal Heave Stability for Braced Excavations in Clays.” Japanese Geotechnical Society Special Publication 2, no. 44 (2016): 1551–1554. doi:10.3208/jgssp.atc6-03.

[16] Pratama, Ignatius Tommy, and Chang-Yu Ou. “Analysis of Sand Boiling Failure in Deep Excavations.” Proceedings of the 2nd International Symposium on Asia Urban GeoEngineering (2018): 125–141. doi:10.1007/978-981-10-6632-0_10.

[17] Fontana, Nicola. “Experimental Analysis of Heaving Phenomena in Sandy Soils.” Journal of Hydraulic Engineering 134, no. 6 (June 2008): 794–799. doi:10.1061/(asce)0733-9429(2008)134:6(794).

[18] Yousefi, Mehdi, Mohammad Sedghi-Asl, and Mansour Parvizi. “Seepage and Boiling Around a Sheet Pile under Different Experimental Configuration.” Journal of Hydrologic Engineering 21, no. 12 (December 2016): 06016015. doi:10.1061/(asce)he.1943-5584.0001449.

[19] Zhao, Guo-qing, Yu-you Yang, and Su-yun Meng. “Failure of Circular Shaft Subjected to Hydraulic Uplift: Field and Numerical Investigation.” Journal of Central South University 27, no. 1 (January 2020): 256–266. doi:10.1007/s11771-020-4293-2.

[20] Pane, Vincenzo, Diego Bellavita, Manuela Cecconi, and Alessia Vecchietti. “Hydraulic Failure of Diaphragm Walls: a Possible Methodology for Safety Improvement.” Geotechnical and Geological Engineering 35, no. 2 (December 27, 2016): 765–780. doi:10.1007/s10706-016-0140-5.
Advanced Design For Manufacturing of Integrated Sustainability “Off-Shore” and “Off-Site” Prototype - MVP “S2_HOME”

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Abstract

The "S2_Home" research project - double safety home - the double safety of living (seismic and social/environmental), pursues the development and research strategy of the De Masi Mechanical Industries of Antonino De Masi, on the themes of innovation related to technologies of automated mechanics, applied to the realization of systems and components at the service of health and quality of life of users. S2_Home pursues the integrated sustainability model between "off-shore" and "off-site" processes. "Off-site" because it applies solutions inspired by robotic automation and advanced manufacturing for the components of a building system between machine shops and off-site. a laboratory for the assembly of systems and services; "off-shore" because it initiates processes of "energy transition" for small and medium-sized user communities. The design process transfers the housing energy-environmental performance of the standard module to the whole integrated supply system, up to the realization of a superior energetic functional model entrusted to the "smart grid". The S2_Home housing module is realized through mobile and self-mounting living systems, that meet the demand for emergency settlements, focusing on the quality of living, the efficiency of operation and usage, and the versatility of construction for different climates and sites sensitive, to the innovation of technological systems and supplies, that are able to characterize the module and make it available to aggregation settlement systems. To realize the economic value through optimizing energy and service operations, as well as the economy of scale on the production chain, using techniques and processes of the company's machine shops.

Keywords: Double Safety; “Off-shore” e “Off-site”; Integrated Design; Prototype/MVP.

1. Introduction

The possible technology and commercial competitiveness of the market to produce "innovation" that is attractive to the demands of users, is the process addressed with an open and progressive approach for the development of the S2_Home – offshore and offshore. The ambition of the S2_Home type is in fact to become a model that triggers an innovation of a “sustainable chain on advanced manufacturing industrialization”. It’s measured on the typological and settlement proposal, able to provide design and technology, innovative products, interceding with a new local market, competitive at national and international levels.

The sustainable project is always, by its very nature, a "total design", able to find moments of compatibility with other structural dimensions of the environment. Such as those more relevant to the landscape and the device's efficiency system, even including its installations. In regimes of hyper sustainability, including short- medium- and long term, and encompassing the verification and measurement of economic feasibility and the evaluation of viable strategies, there is no longer a real need to distinguish between a sustainable approach to the project and a
conventional way of thinking about cities, buildings, and innovative living service products. This is a historical moment in consciousness that presents the only possible way to limit the consumption of exhaustible resources, to renew those of the local and super-local biophysical system. Thinking univocally about the same system’s capacity to produce a comprehensive quality of life, in physical, material and performance aspects, for the environment and other non-urban areas. The question of “total quality” is answered by the definition of "total design": ‘using advanced techniques involving preplanning. One must know what has to be done before doing it, so that one can organise the work properly. The key to a building operation ought to be what one might call the total design - which foresees and plans every stage in the operation. This total design must nowadays be the work of many people, each contributing his particular knowledge, and this team should be controlled by a strong leader, now an executive team with power to decide priorities and reconcile conflicting claims"[1]. We adopt the term “advanced design” rather than project, in order to favour a particular approach. As a concept related to innovation, a design-driven formula, which provides the necessary space for interpretation and communication, in the projected life context, between those who must bear the fruit of the project in its production and those who must use and consume them in post-production. The collective laboratory for each process is defined as a "design discourse", in which those who operate the project always assume the role of interpreter/researcher and give various contributions. "Companies that produce design-driven innovations place great value on their interactions with the interpreter network. These companies know that they are immersed in a collective research laboratory through which companies, designers, artists and schools are carrying out their exploration. These researchers are involved, explicitly and implicitly, in an ongoing dialogue: they exchange insights, interpretations and proposals in the form of studies, conversations, prototypes, images and products. They test the robustness of their assumptions and share their visions. This process of 360-degree research into the possible meanings of things is design discourse” [2]. For this reason and as discussed, the declination of the project is addressed whilst considering the characterizations and reasons of the "advanced design form". The definition of the equivalent adjective announces its purpose: impact design, high performance design, and cybernetic design. It works to trigger a projective process, which can translate procedures of “total design” and is comparable to proposed sustainable inter-scalar operations that have the ability to control the same definitions of the project [3]. With the aim of configuring a possible commentary on the quality produced in architectural landscapes, a conscious use of technologies can enable an understanding of the functioning life-cycle and the relational energetic conditions among all the biophysical and socio-economic systems that react to a certain "level of turbulence". This is indicative of all building environmental dimensions involved in fast, innovative, complex projects.

The "total sustainable design" is therefore charged with a methodical task of knowing how to effectively manage innovation through a generative, multi-level design model (from strategic planning to the open design of the system/object). To govern the ways in which the activities of the processes affect the technical and performance aspects and to express all levels of useful knowledge about the operating systems involved. Such a definition allows the project to assume an “exploratory”, multidisciplinary, non-sectoral or specialized character, but even more a narrative and dedicated role to illustrate paradigmatic issues of interest, of which a possible definition guided by innovation must be found. An approach that although it preserves that projective character of the project, by axiom it leaves room for a fertile state of design thinking, even without its need to achieve a transformation of the building environment. "On the one hand, the project, precisely as an anticipation, refers as a temporal horizon to the future: that is, the project is inseparably linked to the affirmation of the idea of the future. On the other hand, the design activity must necessarily confront the category of the possible. This clarification puts the project away from deterministic interpretations, but at the same time leaves room for the possibility of thinking about design activity outside of an effective desire to transform the present” [4].

The mission of the company that commissioned the project-research is not limited at the realization of patentable prototypes nor the results of "experimental development". The project intends to create industrial systems able to position themselves on innovation markets thus activating innovative sectors on the issues of environmental and social security, in accordance with the rules and processes typical of manufacturing industrialization to other content of innovation and enabling technologies. Moreover, the level of specialization necessary to realize the S2_Home, is realised through works in the machine shop. To transfer that typical approach of innovation processes, we are able to experiment but also technical practices, thanks to the transfer of knowledge of its operators at all levels.

The S2_Home research project is described in the paragraphs of this study, presenting the research methodology (Sec.2), on the themes of design driven innovation, with the measure of innovation of the research project S2_Home: self-assessment of the TRL. The results and discussion (Sec.3) of scientific research activities are proposed in their ability to respond to the production demand for the prototype of the housing module, in all its design levels and definitions of sustainable building performance. In conclusion (Sec.4), a review of some of the essential characteristics, in terms of advancement for experimental research, measured through two aspects: the integrated design and sustainability assessment and the facilitated building site.
2. Research Methodology

2.1. The Customer's Request

The housing module S2_Home, is then realized through the thorough study of mobile and self-elevating systems to answer to the growing housing demand for the emergency settlements or in any new living scenarios. Aiming for a high quality of living, the versatility of the building and it’s efficiency of operations and usage allows it to be located in different climates and other sensitive sites. The project is always responding to all levels of necessary requirements namely the innovation of technological systems and supplies thus characterizing the module and making it available for aggregative systems in geographically different scenarios and specific responses of the module’s performance.

This is the economic strategy for realization made possible through optimizing the processes of thus saving energy operations and services for the module, as well as using techniques and possible processes in the company’s machine shops can produce a feasible and sustainable economic scale. The necessary properties of the module is to have a sustainable and effective means of mobility. The S2_home can be transported using everyday trucks because it’s has the same dimensions as a shipping containers.

![Figure 1. Ordinary transport on three-axis trailer and opening diagram of the modules (Design: Procopio 2017)](image)

Taking into consideration the advancement and uploads of aluminium frames by Safety Cell (patent by De Masi), we have used envelope and steel panels for the configuration of the structural box.

![Figure 2. Envelop, structure and coating system (Design: Sgaramella and Procopio 2017)](image)

2.2. S2_Home – Design Driven Innovation

S2_Home is the product of a high-specialization innovation and development process that has identified three important steps for the definition of activities and products:

Step 1: Concept & Innovation Process/design Project

- Definition of innovative concept and process/project illustration;
- Activities of technical-design definition (architectural and functional aspects; technological and systems-engineering; aseismic construction; pre-engineered and engineered);
- Pre-manufacturing Activities (selection of supplies and preparation of site/prototype activities).
Step 2: Manufacturing and Marketing

- Prototyping and Simulations (construction site);
- Post-production Activities (technical and commercial information);
- Communication and marketing activities on a prototype (on programme).

Step 3: Branding and dissemination

- Experimental development and market positioning of the commercial prototype (patent);
- Dissemination and industrialization programme.

The transition between step 1 and step 2, “from concept to prototyping”, has affected the engineering activities of the project, which see their conclusion in July 2019, then start the activities in the workshop with the realization of the prototype/MVP and are scanned in two times:

TIME I: A. Concept and Innovation Design (definition, project, pre-manufacturing); B. Manufacturing Envelope; (Prototyping, envelope testing, manufacturing); C. Report and dissemination results.

TIME II: D. Project revision and selection of the typological module I phase; E. Engineering of the structural module and the hybrid systems; F. Engineering of the project with drawings of factory and fabrication; G. Process engineering with pre-prototyping and eco-design, components, models/manufacturing in the company and sensoring testing for envelop and skin; H. Report and dissemination results.

These activities, in fact, are built on the prototype model chosen that corresponds to the type MVP-minimum viable product: “(…) an MVP allows you to accelerate your learning about a possible solution whilst using minimal resources. It does this by testing only the essential core of your concept (rather than the full solution) with real users in practice. This means that you can find out early on if there is an actual need or demand for the solution, what is working and what isn’t, and make any adjustments accordingly (this is called pivoting in the lean-startup scene). MVPs are often associated with technology, and aren’t currently common in public innovation, but may have great potential for situations that deal with a fast-paced political development cycle or require ongoing improvement of public services and public policies. MVPs are about using fewer resources and minimal effort to gather insights and obtain feedback on potential changes” [5].
Home S2 as prototype-MVP wants to position its solution in the innovation market with one of its possible configurations (85 sq. m.), opening the solutions foreseen for the integration of innovative products on some manufacturing characteristics and non-structural affecting systems, envelope coatings, enabling technologies to increase the energy-environmental reactivity of the skin of the envelope (reactive skin/cybernetic skin). Moreover, this possibility allows to act by integrating new components, as on a catalog system and it is realized, for the planned configuration of its aggregative scenarios, different for climate conditions, landscape, utility, functionality, energy efficiency, and durability.

![Figure 5. Modularity of different types and distribution on two floors (Design: Sgaramella 2017)](image)

2.3. The Measure of Innovation of the Research Project S2_Home: Self-Assessment of the TRL

Among the ambitions of the research project for the S2_Home module, that of positioning all the work produced in the innovation ecosystem, in which the exploitation of the results of scientific research can take place through an industrial validation process, with a definition of the process of high added value industrialization. As anticipated in the introduction, this work, moreover, can be placed in specialist areas related to the Industry 4.0 system, referring to the Horizon 2020 ket's to process/product technologies on "advanced manufacturing systems" and "advanced materials" and in the production context of Strategy Regional Smart - S3 Calabria, for the trajectories of "Sustainable Building" and "Smart Manufacturing".

The measure of innovation of the research project by the team that operated in phase I and II, can be tested on an evaluation method that refers to the concept of "technological maturity", testing the process as defined by the TRL, Technology Readiness Level "on a scale of values from 1 to 9, as specified by the European Commission in the Program Horizon 2020 – Work Programme 2018-2020 General Annexes – Extract from Part 19 – Commission Decision C (2017)7124.

The references on which to compare their applications to evaluate this level of innovation are:

- TRL 1 – basic principles observed
- TRL 2 – technology concept formulated
- TRL 3 – experimental proof of concept
- TRL 4 – technology validated in lab
- TRL 5 – technology validated in relevant environment (industrially relevant environment in the case of key enabling technologies)
- TRL 6 – technology demonstrated in relevant environment (industrially relevant environment in the case of key enabling technologies)
- TRL 7 – system prototype demonstration in operational environment
- TRL 8 – system complete and qualified
- TRL 9 – actual system proven in operational environment (competitive manufacturing in the case of key enabling technologies; or in space)
Table 1. Details of the S2_Home activities with reference to the TRL model are given below

| TRL | Step S2_Home | Advanced Design S2_Home |
|-----|--------------|-------------------------|
| TRL 1 – Basic principles observed | 1° step | Process / project construction: design driven innovation.  
- Process innovation from proof of concept to MVP prototype  
- Innovation between executive design and factory design  
- Innovation of the compatibility of the materials for the envelope towards the prototype |
| TRL 2 – Technology concept formulated | 1° step | Concept and design (1):  
- Integrated off shore and off site sustainability  
- Housing module between architecture and landscape |
| TRL 3 – Experimental proof of concept | 1° step | Concept e design (2):  
- Energy model and energy-environmental performance for climate scenarios  
- Structural model and its typological-technological configuration  
- Design, technologies and materials of the mounting systems: casing; coverage  
Feasibility scenarios:  
- Construction of the smart grid at the settlement aggregation scale: systems, operations, architecture and network performance efficiency  
- Definition of aggregation scenarios and environmental landscape performances  
- Definition of cost scenarios and comparison with similar innovative products, assessment of impacts and conveniences |
| TRL 4 – Technology validated in lab | 1° step | To Officine De Masi, Laboratori Enea Trisaia, PMopenlab  
- Realization of the panel module at the factory  
- Testing and aging on the scale panel module, discussion of the results  
- Modeling and automation design of the panel module and the housing module |
| TRL 5 – Technology validated in relevant environment (industrially relevant environment in the case of key enabling technologies)/ | 2° step | Product and process technologies: engineering phase  
- Executive drawing of the envelope and the buffer system  
- Executive drawing of the plants on the hybrid model  
- Executive design of structures, assembly and automation systems  
Process technologies: assessment of sustainability  
- Smart grid requirements and type of settlement models in transition  
- Design and evaluation of the sustainability performances of the cases on the scenarios Start of industrial validation process. |
| TRL 6 – Tecnologia convalidata in ambiente industrialmente rilevante | 2° step | To PMopenlab, Officine De Masi  
Integreted Design and Maufacturing:  
- Ecodesign, Modeling and Control of the technical solution in pre-prototyping  
- Energy-environmental performance monitoring and cybernetics with Arduino technology on the facade system: pre-feasibility |
| TRL 7 – System prototype demonstration in operational environment | 3° step | Prototyping phase to Officine De Masi:  
- verification of the engineering phase and factory drawings: technological definition of the S2_home module |
3. Results and Discussion

3.1. S2_Home – Concept of Integrated Sustainability “Off-shore” e “Off-site”

S2_Home pursues the model of integrated sustainability between "off-shore" performance and "off-site" process. High standards of energy efficiency through “offshore” models capable of making settlements autonomous and starting of "energy transition" processes for small and medium-sized user communities and advanced "off-site" building processes that realize all the components of a system construction between machine shop and off-site and reduce the site a workshop for the assembly of systems and services. But "off-site" also means interest in design processes with digital control and experimentation with industrialization 4.0, with solutions and applications inspired by robotic automation and advanced manufacturing.

The "offshore" system is realized through the design process that transfers the energy-environmental performance of the standard module and entrusts the technological characteristics to its envelope, to the entire integrated system of high efficiency supplies up to the realization of an energy functional model higher level with the "smart grid". This system becomes the guiding matrix for the organization proposed with the multi-scenarios realized through the housing of the S2_Home type, but also with the network of energy services and landscape corridors, which produce outdoor spaces and the quality at the urban scale. Also in the proposal S2_Home module, this process highlights its economic and environmental affordability, its competitive ability to be realized the innovative construction industry and the green yard.

The construction site-laboratory as a place that carries out some of its activities already in the construction phase of open prefabricated systems, to then find the assembly stages on the site, also meets the speed up the realization of the works and of configuration of settlements (especially for emergency scenarios), as well as the ability to check out-of-built the integrated energy services and immediately conceive them as a condition of quality and effective functioning of the entire system building. Furthermore, the level of specialization necessary to realize the S2_Home type, through works of realization in the machine shop, transfers the typical approach of innovation processes, capable of becoming experimental practices but also a new process of products "made in Calabria", thanks to the transfer of the knowledge of its operators at all levels.

3.2. Performance Levels

The project-design of the housing module has transferred the requests of the client (performance on the application) into the integrated concept of S2_Home, with a process of meta-projective process that has become the program of the study activities and the reference of integrated design for the transition from the "concept" step to the "definition of the type" step. Already in the concept phase the planning in started for a logic about the spaces-environment; of the filter spaces; of the logic of the structure; of the logic of the envelope; about the logic of the roofing (performance on the definition of integrated requirements).

CONCEPT INTEGRATO

Figure 7. Integrated Concept (Design: Nava 2017)
Also in this experience it is a matter of proceeding according to the trajectory of sustainability served by a "total design" approach. The sustainable project is always a "total design" capable of rediscovering moments of compatibility with other dimensions of the structures of the environment, those more related to the landscape and to the devices for the efficiency of the systems, to its plants, up to the verification and measurement of the economic feasibility and evaluation of the strategies that can be pursued, in short-term and long-term and medium-term hypersustainability regimes. There is no longer a real need to distinguish between a sustainable project approach and a conventional way of thinking about settlement systems, buildings, and the innovation of living service products [6, 7].

3.3. Level 0: Performance on the application of the prototype/MVP

Through the level 0 design process, the targets of requirements are defined on the prototype/MVP application for:

- **A structure for sensitive contexts**: beyond an "emergency" structure for a housing system for "sensitive contexts" and "off shore", responding with the transition or the energy-environmental autonomy, in addition to the settlements already served by networks.

- **A flexible modular living system**: according to requests of the clients referable to the structural aseismic project and to the morphological and distribution type of the environments, for a different type of use (couple, family, disabled/elderly) with a basic module 55 sqm and making types from mq. 85/112/170 to one/two floors.

- **For a network metabolism**: with a module of a settlement system that optimizes some relationships with the networks, while configuring its corresponding aggregation, with the possibility of having architectural and operating variants (districts in transition and smart grid operation).

- **A competitive economic system**: a sustainable system also from an economic point of view with the cost of the fully equipped basic module, from a minimum of Euro 1400/sqm to a max of 1600 Euro/sqm.

- **Through an advanced industrialization – Advanced metalmechanics and sensoring**: with a module made from an experimental prototype/MVP on a project designed for its industrialization with a dry construction system and innovative reactive envelope systems, with the possibility of integrating Arduino technologies to the skin of the casing. A transportable kit system, to be assembled in situ completely, with stages in self-assembly partly with automation and partly in operational assembly.

![Figure 8. Built-in "off-site" assembly systems. (Design: Astorino and Procopio)](image)

The structural box, entirely in steel type S 235, are made with four corner posts of adequate section to which are connected some transverse beams (arranged in correspondence of the floor of trampling and covering). Statically, the structure is of the single-span frame type, in both directions, which will be placed on a foundation structure, also in steel, specially sized according to the destination site. In the structural sizing, carried out in compliance with the technical standards in force, in addition to the permanent, structural and no permanent actions, the effects produced by the earthquake, wind and snow loads were taken into account. The connections between the structural elements are of the type welded with a corner bead [8].

3.4. Level 1: Performance on the Integrated Requirements Of Safety, Energy And Material Innovation

Through the level 1 design process, we define the targets of integrated requirements for safety, energy and materials innovation for

- **A Kit, safe system “agile to assemble”**: with a box mountable with wall sections and shelves and integrated systems, such as an intelligent "kit" of structural wall sections, partition walls and closing on the box-type structure, made of steel and aluminium. With automation systems on the elements that cannot be moved manually.
and reduced connection systems to the maximum (low maintenance). An anti-seismic module on support feet of the entire housing module, on a base prepared for the purpose.

- **Integrated roofing system:** a structural cover, which can be integrated for solar and photovoltaic technologies, and which can be oriented with mechanical handling systems and more efficient orientation.

- **A functional base plate:** with a foundation plenum and support system for the assembly of the structure and the closing border, able to contain all the systems, complete with all the equipment also for monitoring and the structural support of the foundation. A network of distribution of the networks to the elevated and supply systems for the services and supplies of the module.

- **A low consumption house:** with a reactive envelope, capable of thermoregulating the module at cold and hot temperatures, sized to perform efficiently for different climatic zones (from A to F) and with the same insulation system, with triple-stratified panels in recycled polyester and with modules inserted in a curtain wall. A model of a home that can be certified energetically and also on some national and European protocols (itaca/leed), a low / zero energy consumption house (energy class A4).

- **A system connected to an energy grid:** for a house powered by solar technologies and water recycling systems (services and kitchen). The coverage of energy needs starts at 96.4% from PV and 3.6% from diesel for solar batteries, with a 2-day module autonomy, isolated from network supplies or smart grid operation. An energy node in a smart-grid, capable of governing the transition of production, storage and distribution for an entire cluster with the 9 living modules connected to the network as energy poles.

- **An ecological, innovative and produced house of a circular economy model:** energetically performing materials, ecological, compatible with a dry construction and coming from recycling processes and chains. The levels of functioning as an isolated system or networked with settlement services, realizes the performance of a system that responds to circular economy models: consumption, production, recycling, innovation. A durable home tested for the impacts of climate change and aging (energy and aging tests conducted by Enea for the Home S2 project).

Figure 9. Envelope System: 1. Indoor cladding (e.g. fire wood panels), 2. Vapour Barrier, 3. Thermo-acoustic insulation in polyester fibres recycled on aluminium frames, 4. Windproof barrier, 5. External skin (e.g. boxed slats in varnished aluminium). Project and assembly prototype for workshop testing De Masì machine shops. (Nava, Astorino and Procopio, 2017).

The building envelope is made up of modular elements (panels) that can be mounted manually through a rail system on the external perimeter of the housing module and then completed with the external and internal coverings. The panel, depending on the position inside the enclosure system (on the shorter or longer side of the housing module), is prepared in two modularity for the width and different for the height based on its specific function (opaque panel, transparent with window, transparent / opaque for door).

The composition of the sub-modules gives rise to a diversification of the morphology of the casing as a function of their combination with the sole constraint of the transparent surface dimension that must not exceed 17 square meters for the 60 square meter housing module of commercial surface (proportional increase for the other compositions of the housing module). The modules consist of a structure in aluminium box to form a frame capable of accommodating the layering (thermo-acoustic insulation panels in 100% recycled polyester fibers and the external wind and inner barrier protections to steam) or the window components.

The vertical closure will then be completed through the laying of the coatings (of various types and materials) inside (for example in wood) and outside (for example in painted aluminium), with morphological and material alternatives on different settlement scenarios. The integrated roofing system is designed not only to respond to the
need for protection from atmospheric agents and from the external microclimate but also to accommodate an innovative system for the integration of photovoltaic panels and solar thermal collectors, mounted with a steel box structure which is laid and fixed in special seats on the finished top in steel corrugated sheet; the technological system, therefore, can be tilted according to the optimal orientation to be given to the solar sensors through appropriate electric actuators [9].

3.5. Level 2: Performance on Multi-scenario Settlement Models: Energy Clusters and Smart Grids

Through the level 2 design process, the classes of requirements (targets) are defined for the multi-scenario settlement models described below.

- **The configuration of an energy cluster.**

The housing module HOME S2 self-sufficient, at the aggregative level configures that an “energy block” or energy cluster, realizing in it the concept of modularity: by combining different blocks / energy clusters you go to compose an aggregative structure at the level of the neighborhood to build pieces of cities self-sufficient and disconnected from network operation; the only network will be constituted by the system of connections between housing modules and between energy clusters with the possibility of being able to be connected to an extranet system (connection to already existing network systems). Isolates dimensioned on the needs of 20,000 KWh, served by micro wind and photovoltaic. Housing systems tested on 360 in / 9 modules, 695b / 17 modules, 220 in / 5 modules, for climate zones from A to F. The energy cluster structures not only energy connections but also those for the recovery of environmental resources (recovery rainwater and / or wastewater) on the model of energy-environmental smart grids (phyto-purification, PV, wind, park green). Furthermore, CO2 storage tanks, as far as each type unit is able to save in energy terms with highly efficient hybrid operating models.

- **Smart grid and energy landscape and possible scenarios.**

These scenarios can also refer to the need to create new neighborhoods and new territories settled in different site and landscape conditions. These are new landscape units, but also new stable environmental systems, capable of producing for themselves all the service they need and thus ensuring users from an environmental and social point of view. But it is also a new settlement geography, able to find optimal localizations from the bioclimatic point of view and better accessibility in their relationship with filter and external spaces.

The references to the different climatic zones, allow to show the performances obtainable in different latitudes, but also to test the typological-technological model of S2_home, conceived as "adaptive" to the different climates, with a reactive shell capable of changing skin and giving itself a new morphological configuration and scenery with the landscape that hosts it, with high permeability settlements. In the multi-scenario functional hypotheses, the aggregations could serve settlements in case of emergency (post-earthquake or migration); districts with the transfer of the built-up area during major urban transformations (post-demolition regeneration plans, new satellite districts); villages for intended use, such as employees in urban construction sites, large and small investments in agricultural production sectors with settlements of communities in transition.

![Figure 10. Smart grid system (Design: Sgaramella 2017) and hybrid housing model (Astorino 2017)](image)

The energy model was designed and dimensioned not only with respect to the concept of off-shore smart grid, but also following the performance and energy and capacity that an entire block must guarantee in terms of energy production, organizing the settlement structure on display of the modules and construction of the smart grid. The compositional settlement model, basically observes two criteria: the minimum distance between the buildings and the north positioning of the two-level modules. A minimum distance of 12 m must be guaranteed for each building. The distance doubles in the case of multi-level modules. The general layout can create a free central space, inside the...
block, which acts as a public area on which to work for the definition of main neighborhood services. For the disposal of waste water, a phytodepuration system will be used whose minimum areas to be guaranteed refer to the number of inhabitants of each module considered. For schematic convenience of representation of the individual modules, in the two settlement compositions the minimum corresponding phyto-purification surface will be shown, next to each module, to be envisaged at a minimum distance of 10 m [10].

![Figure 11. Vision Agro-scenarios (Design: Zinghini 2019)](image)

4. Conclusions

In conclusion, a review of some of the essential characteristics of the concept-idea, which in the integrated design phase are defined and designed to verify their feasibility, in their transfer from "demand for expected performance" to "typological, technological choices", skills that they must be transformed into attributes and characteristics for systems and components and in their compatibility with the industrialized processes of factory and construction site.

4.1. Integrated Design and Sustainability Assessment

Already from the concept phase towards integrated design, it was possible to translate the express needs for the qualification of the housing module with double environmental and seismic safety, into classes of corresponding requirements, through the analysis and development of technological systems, through work evaluation of elements and parameters to arrive at the definition of the type selected for the purpose of manufacturing the prototype / MVP. All the concepts examined were "multidimensional", like all evaluations sought through strategies used in their complexity. Each decision, on structure, materials, technologies, was the result of considerations translated into a design dedicated to the realization phase in the workshop and on the construction site.

“(…) the ability to define the objectives and requirements of a project, include the relevant aspects and assess the interactions between them, calls for different perceptions and ways of seeing the issues from everyone involved in the process. The challenge in controlling planning and design is to bring together the different points of view arising out of the different perceptions at the problem (…)” [11].

Through the "off-site" construction methods, the construction site becomes lighter, an experimental laboratory in which to tackle the assembly and assembly activities of the designed systems. The relationship between factory and site has direct repercussions on quality, comfort and safety. Likewise, the engineering process down to factory drawings approaches a concept of advanced industrialization, which has a lot to do with digital and manufacturing, for the control of technical solutions [12-14]. The five recognizable elements for off-site construction are also guiding trajectories for the S2_Home project: digital infrastructure, productivity, different production scales, environmental sustainability, quality and safety together. Last but not least, in terms of the industrialized production chain, the reliability of the products, the traceability of the components, the programmable maintenance, as well as the containment of energy costs. They become very important targets for controlling the life cycle of the housing module such as its economic sustainability over time.
4.2. Facilitated Building Site

With reference to the construction site, the design had to reconnect the requirements for transportability, with the definition of the casing systems, joints, structures and systems dimensioned on the 85 square meter module on one level. On the need to think these systems as aggregated in the workshop, to be transported and assembled on site. The modularity of systems has pursued a medium level of unification, because the implementation of the solutions, for example on the functioning of the hybrid energy-environmental model, have however required a diversification between the fronts and therefore a consequent diversification of the systems of the shell structure and the relationship between opaque and transparent surface, as well as for the type of fixtures. Despite this necessary expression of quality, it can be said that the control of the integrated system design and the off-site in the workshop make operational assembly easier on site.

Also the research project Home S2, starts an interesting field of experimentation on the themes of the relationship between process innovation concerning the circular economy and project innovation, through enabling technologies, for the definition of functioning models based on smart technology grid. In this theoretical and application scenario, the Home S2 module certainly takes on a value that exceeds the size of the product and expresses all its innovative features as an agile and active resilience device for the settlement scenarios to which it provides its service. “(...) advanced design is a double-bind environment, characterized by contrasting aspect, like two faces of the same coin, which show different ways of tackling the project. In this double connotation, there is a shared root: an Advanced Design which is synonymous with innovation” [15].

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6. Conflicts of Interest

The authors declare no conflict of interest.

7. References

[1] Ove Arup. “Teams for Total Design”, in Chipperfield. David, Long K., Bose S. “Common Ground”, Marsilio edizioni, Venezia, (2012):129-131.

[2] Verganti, Roberto. Design-Driven Innovation–Cambiare le regole della competizione innovando radicalmente il significato dei prodotti e dei servizi. Etas, Milano, (2009).

[3] Nava. Consuelo, “Edifici Sostenibili”, DEI edizioni, Roma, (2012).

[4] Campioli. Andrea, “Le culture del progetto”, in Nardi G., (a cura), “Aspettando il progetto”, F.Angeli ed., Milano, (1997).

[5] Leurs. Bas, Duggan. Kelly, 2018, “Proof of concept, prototype, pilot, MVP what’s in a name”. Available online: https://www.nesta.org.uk/blog/proof-of-concept-prototype-pilot-mvp-whats-in-a-name/ (Accessed on 15 July 2020).

[6] Nava. Consuelo, “Design Driven Innovation “off-shore” e “off-site” in SID Series, Aracne edizioni, Roma, (2019).

[7] Nava. Consuelo, “S2_Home – concept della sostenibilità integrate “off-shore” e “off-site”, “Design Driven Innovation “off-shore” e “off-site” in SID Series, Aracne edizioni, Roma, (2019).

[8] Astorino. Francesco, Procopio A. “Il Disegno esecutivo delle strutture, dei sistemi di assemblaggio e automazione, parte V, “Design Driven Innovation “off-shore” e “off-site” in SID Series, Aracne edizioni, Roma (2019): 124-129.

[9] Nava. Consuelo, Astorino R., Procopio A., “Realizzazione in fabbrica del pannello e del modulo abitativo”, part II, “Design Driven Innovation “off-shore” e “off-site” in SID Series, Aracne edizioni, Roma (2019): 53-57.

[10] Nava. Consuelo, Sgaramella G., 2017, “Definizione degli scenari aggregativi e prestazioni ambientali e di paesaggio”, parte III. “Design Driven Innovation “off-shore” e “off-site” in SID Series, (2019): 84-89, Aracne edizioni, Roma.

[11] Drexler. Hans, El Khouli. Sebastian, “Holistic Housing – Concepts, Design Strategies and Processes”, Detail ed., Munich, (2012).

[12] Nava. Consuelo, “Ipersostenibilità e Tecnologie abilitanti:Teoria, Metodo e Progetto”. Applicazione V, Aracne ed., Roma, (2019): 625-640.

[13] Onyelowe, Kennedy C, George Alaneme, Duc Bui Van, Manh Nguyen Van, Charles Ezugwu, Talal Amhadi, Felix Sosa, Francis Orji, and Benjamin Ugorgi. “Generalized Review on EVD and Constraints Simplex Method of Materials Properties Optimization for Civil Engineering.” Civil Engineering Journal 5, no. 3 (March 19, 2019): 729. doi:10.28991/cej-2019-03091283.

[14] Khorramshahi, Mohammad Reza, and Ali Mokhtari. “Automatic Construction by Contour Crafting Technology.” Emerging Science Journal 1, no. 1 (July 8, 2017): 28. doi:10.28991/esj-2017-01113.

[15] Celi. M., “Advanced Design Cultures. Long-Term Perspective and Continuous Innovation”, Springer, SW, (2010).
Effect of Changing Properties of Wythes in Precast Structural Sandwich Panels

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Abstract
This study investigates the effects of changing in the properties of face and core wythes in structural sandwich panels (with dimensions of 500×500 mm and 120 mm total height). Concrete face wythes of three grades (80, 70, 37) MPa, thicknesses of (25, 35, and 45) mm, and three types of core materials (high density foam, polyethylene foam, and palm bark) were used in the production of panels. Steel shear connectors were installed in the panels with angle of 45°. Three-point bending load test was carried out on all panels and results were compared with both of the theoretical extremes capacities of non-composite and fully-composite states and ANSYS software results. The degree of composite action (%) and the (strength/weight) ratio were the main parameters that judged the specimens. It was found that upgrading concrete increased overall strength of slabs especially in high strength concrete (80 MPa), however the use of lightweight concrete (70 MPa) caused high (strength/weight) ratio due to very lightweight. Results revealed that decreasing thickness of concrete face wythes had a positive effect on strength/weight ratio (although the ultimate loads decreased) that enhanced the performance of panels as lightweight structural panels. The optimum face wythe thickness is that of 2.5 cm and has high (strength/weight) ratio. It was noticed that adding polyethylene foam as a core material results in positive effect and high (strength/weight) ratio. Results revealed that high strength concrete (80 MPa) and light-weight concrete (37 MPa) are very successful in the production face wythes of precast light-weight sandwich panels that can obtain high (strength/weight) ratio and high percent of composite action.

Keywords: Sandwich Panel; Composite; Light-Weight; Strength/Weight; Insulation Core.

1. Introduction
The common typical cross section of structural precast sandwich panel (SPSP) consists of two face wythes of concrete separated by core wythe that almost consists of heat insulation material [1]. Various parameters control the structural behaviour and the overall ultimate capacity of structural precast sandwich panels such as the grade of concrete in concrete face wythes, thickness of concrete face wythes, distribution of shear connectors, rigidity of core material, and bond between core-concrete interfaces [1, 2]. It was found that shear flow capacity tends to decrease with the increase of core wythe [1]. Previously, ultra-high performance concrete was successfully the production of face wythes of SPSP with the core material of high performance expanded polystyrene. The panels were classified as light weighted high-strength structural sandwich panel [2]. The relative light weight comparing to high strength is playing the main role which controls the performance of SPSP [2, 4].

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Many types of insulation materials were tested as core materials in SPSP. It was found that expanded polystyrene foam was very successful at service load level [3]. It was also found that high density foam is very practical, rigid, and light-weight when it was used as the core material in SPSP [14, 16]. There is more than material to be used as shear connectors in SPSPS such as steel, carbon fiber, and basalt fiber reinforcement polymers (BFRP) that can play this effectively [4, 5], however the composite action obtained with BFRP was less than that one with steel shear connectors [5]. Generally, it was found that fiber reinforced polymers are successful to be used in structural sandwich panels as main steel and shear connectors due to its increased deflection and deformability [5, 6]. The increases of the number of shear connectors was found to increase the overall ultimate capacity of panels [4, 6, 16]. Sudden failure was observed in four-point loading system test due to the Combination of flexural and shear stresses [5, 7]. It was previously found that the lightweight in sandwich panels gave opposite advantages in overall capacity loading [8, 13].

It was noticed also that the overall deflection of sandwich panels depends on the shear deformation of the core layer, material, and its thickness [5, 9]. It was found also that the truss-shaped connectors made of wires were successful to achieve composite action [10]. The continuous-type connectors can structurally behave better than stud-type connectors [11]. In general, also it was noticed that larger deformation was observed due to the increased core thickness [12, 13]. Many software like ABAQUS software were used to analysis composite sections [17]. It was found previously that increasing the thickness of lower angles in connection in composite sections caused an increase in energy dissipation of connections and a decrease in the induced initial stiffness from post-tensioned strands [17].

This article consists of: research significance, numerical modeling, experimental part, results, and conclusions. Results obtained from experimental tests were analyzed and compared with the predicted ultimate loads. Results also were obtained from ANSYS software and the theoretical extremes of both of states: fully-composite and non-composite. The investigation of changing properties of wythes is detailed in discussion section to determine the optimum concrete grade, concrete thickness, and core material. The following figure shows the flow chart of methodology of this research.

![Figure 1. The flow chart of Study](attachment://flow_chart.png)
Structural lightweight precast sandwich panels were used worldwide latest decade. It can provide the optimum solution between weight, heat insulation, and strength. Its efficiency depends on the degree of composite action achieved by structural behavior in case of loading, light weight, and the ability of heat insulation [14, 16]. Therefore, it was increasing demand worldwide to determine the optimum composite system of structural sandwich panels. Many parameters can be changed to enhance the structural behavior of panels. In this research, the input parameters were: the type of outer face concrete wythes, the thickness of them, and the type core insulation material. Those parameters were changed to achieve the optimum incorporation components that approach composite action [4]. Core wythe should be taken into consideration to perform the role of it: heat insulation and lightweight [13]. Strength\weight ratio should also have studied to get effective parameter that can judge the panels.

2. Numerical Simulation

The predictions of failure capacities were obtained from ANSYS 18.2 software. Mesh properties were adaptive size function with the element size of 95 mm and with the coarse relevance enter with minimum edge length of 18.85 mm as shown in Figure 2. Materials were defined in engineering data was concrete (with edited density and compressive strength of each concrete type) and steel (with fatigue data at zero mean stress comes from 1998 ASME BPV code, section 8, Div2 Table 5-11-1). Then the model was solved with the software to get the ultimate load. The ultimate loads obtained from the software were compared with the experimental ultimate loads. Figure 15 shows the equivalent stress in numerical modeling and Figure 16 Shows the equivalent elastic strain in numerical modeling. Failure shape and deformation of numerical modeling are shown in Figure 13. The predicted overall ultimate failure loads from the software are shown in Table 6.

3. Objectives and Items of Investigation

It was noticed that the efficiency of precast sandwich panels depends also on its weight and the ability of heat isolation and overall strength [2, 3]. Therefore, the main objectives of this study can be summarized as the following: obtaining a high (strength\weight) ratio by increasing strength of slabs with a lighter weight of slabs, approaching fully-composite action as possible by choosing the optimum type, the thickness of concrete, and core material. The ultimate load obtained from experimental tests was compared with the theoretical extreme capacities for each slab (100% degree of composite and 0% degree of composite) in the form % composite.

Fully composite state occurs when the specimen behaves a solid slab with total thickness of 120 mm. Non-composite state was obtained when each concrete wythe behaves alone. The main role of shear connectors is transferring shear between concrete wythes. Changing variables causes occurring of composite action with different degrees. Extreme capacities in both states are shown in Table 6. Calculations of capacities in both states [15] can be detailed as following equations from (1) to (11) [4, 15]. Comparisons between specimens were performed to determine the optimum components. Used parameters were concrete grade, concrete thicknesses and core material.

\[
As = 169.56 \text{ mm}^2 \tag{1}
\]

\[
Fs = As \times fy = 47476.8 \text{ kN} \tag{2}
\]

\[
a = \frac{(As \times fy)}{(0.85 \times Fcu \times b)} \tag{3}
\]
a) Non-composite state: (for thicknesses of 25, 35 and 45 mm)

\[ x = \frac{a}{0.9} \]  \hspace{1cm} (4)

\[ d_1 = \frac{(Thickness \ of \ one \ concrete \ wythe)}{2} - \left(\frac{x}{2}\right) \]  \hspace{1cm} (5)

\[ Mu = 2 \times ((As \times Fy) \times (d_1 - \left(\frac{x}{2}\right))) \]  \hspace{1cm} (6)

\[ Pu = \frac{9 \times Mu}{L} \]  \hspace{1cm} (7)

b) Fully-composite state: (for thickness of 120 mm)

\[ x = \frac{a}{0.9} \]  \hspace{1cm} (8)

\[ d_1 = \frac{(120 \ mm)}{2} - \left(\frac{x}{2}\right) \]  \hspace{1cm} (9)

\[ Mu = ((As \times Fy) \times (d_1 - \left(\frac{x}{2}\right))) \]  \hspace{1cm} (10)

\[ Pu = \frac{(8 \times Mu)}{L} \]  \hspace{1cm} (11)

Where:

As = Area of tension reinforcement;

b = Per meter length of panel;

fy = Steel yield stress;

Fs = Force in tension reinforcement;

Fcu = Concrete characteristic strength;

a = Depth of neutral axis measured from the more highly compressed face;

Mu = Ultimate moment capacity under flexural;

Pu = The total load resisted by the panel;

d1 = Depth of the neutral axis.

4. Experimental Program

Seven specimens were prepared in the experimental program. The size of all specimens is 500×500 mm with total height of 120 mm. Three grades of concrete (80.3, 70.4, and 37) MPa, three thicknesses of upper and lower concrete wythes (25, 35, and 45 mm), and three types of core materials (high density foam, polyethylene foam and palm bark) were used in production of specimens. Each specimen consists of upper and lower wythes of reinforced concrete (5Φ 6mm per meter length) plain steel reinforcement in both directions in the middle of concrete wythes and core insulation material. In each specimen, 16 pieces of steel shear connectors with diameter of Φ12 mm were installed with angle of 45° in its specific positions. The total contents of specimens is shown in Table 1.

| No. | Concrete Type | Concrete Thickness (mm) | Core Type | Core Thickness (mm) | Shear connector Type, Diameter (mm) |
|-----|---------------|------------------------|-----------|--------------------|-----------------------------------|
| 1   | HSC           | 45                     | HD Foam   | 30                 | Steel Φ (12)                      |
| 2   | LHSC          | 45                     | HD Foam   | 30                 | Steel Φ (12)                      |
| 3   | LHSC          | 45                     | PE Foam   | 30                 | Steel Φ (12)                      |
| 4   | LHSC          | 45                     | Palm Bark | 30                 | Steel Φ (12)                      |
| 5   | LWC           | 45                     | HD Foam   | 30                 | Steel Φ (12)                      |
| 6   | LHSC          | 25                     | HD Foam   | 70                 | Steel Φ (12)                      |
| 7   | LHSC          | 35                     | HD Foam   | 50                 | Steel Φ (12)                      |
4.1. Materials

Ordinary Portland cement (OPC) CEMІ 42.5N, silica fume, and quartz powder were used in the production of concrete wythes. Three types of core insulation were used: high density foam, polyethylene foam, and palm bark. The specific gravities of the materials are listed in Table 2. A polycarboxylic ether based superplasticizer complying with ASTM C494 (Type G) was used in the production of concrete. The chemical analysis of cement and silica fume and quartz powder are listed in Table 3. River sand, dolomite, and Leca coarse aggregates were used. The characteristics and the ingredients of three grades of concrete are listed in Table 4. Plain mild steel was used in the main reinforcement of concrete. Steel shear connectors with diameter of steel 12 mm were used. The specified tensile strength of steel is in Table 5.

Table 2. The specific gravity of the materials

| Material            | Specific gravity |
|---------------------|------------------|
| Sand                | 2.66             |
| Cement              | 3.15             |
| Dolomite            | 2.60             |
| Quartz powder       | 2.70             |
| Silica fume         | 2.15             |
| Leca aggregates     | 2.42             |
| Superplasticizer    | 1.15             |

Table 3. Chemical analysis of cement, silica fume and quartz powder

|                      | OPC       | Silica fume | Quartz Powder |
|----------------------|-----------|-------------|---------------|
| SiO₂, %              | 21.58     | 96.02       | 0.97          |
| Al₂O₃, %             | 4.94      | 1.01        | 0.83          |
| CaO, %               | 61.09     | –           | 1.02          |
| MgO, %               | 1.65      | 0.18        | 0.21          |
| SO₃, %               | 3.22      | 0.26        | 0.33          |
| Na₂O, %              | 0.5       | 0.14        | 0.05          |
| K₂O, %               | 0.18      | 0.35        | 0.22          |
| Fe₂O₃, %             | 3.56      | 0.52        | 0.35          |
| Cl₂ %                | –         | 0.16        | 0.05          |
| Loss of ignition, %  | 2.60      | –           | –             |

Table 4. Characteristics and the ingredients of three grades of concrete.

| Concrete type               | HSC (High strength concrete) | LHSC (Light high strength concrete) | LWC (Light weight concrete) |
|-----------------------------|------------------------------|-------------------------------------|-----------------------------|
| Characteristic strength (MPa)| 80.3                         | 70.4                                | 37                          |
| Density (kg/m³)             | 2660                         | 2440                                | 1800                        |
| Cement (kg/m³)              | 450                          | 950                                 | 555                         |
| Sand (% of cementitious)    | 115%                         | 66%                                 | 60%                         |
| Dolomite (% of cementitious)| 300%                         | 0                                   | 0                           |
| Leca aggregates (% of cementitious) | 0                          | 0                                   | 65%                         |
| Quartz Powder (% of cementitious) | 0                          | 23%                                 | 0                           |
| Silica fume (% of cement)   | 0 %                          | 30%                                 | 42%                         |
| Superplasticizer (% of cementitious) | 2%                         | 3.8%                                | 2%                          |
| Water (% of cementitious)   | 19%                          | 17.2%                               | 39%                         |

Table 5. Characteristics of steel shear connectors

| Characteristic                     | Steel Bars |
|------------------------------------|------------|
| Dimensions in. (mm)                | Bars with diameter (12) |
| Specified tensile strength (kN/mm²)| 0.8        |
4.2. Preparing Specimens

The sequence of preparing specimens is shown in Figure 3. Shear connectors were installed in its specific locations shown in Figure 4. Cross section for one specimen is shown in Figure 5.

Figure 3. Sequence of preparing panels

Figure 4. Distribution of shear connectors

Figure 5. Cross section for one specimen
4.3. Items of Investigation

Seven specimens were tested under flexural load by three-point loading system as shown in Figure 6. The weight of each specimen was obtained with calibrated weight scale to get strength/weight for each specimen. Results and strength/weight are shown in Table 7. Characteristic strength and density for each grade are shown in Table 4.

![Figure 6. Flexural test at laboratory](image)

5. Results and Discussion

5.1. Results

The ultimate strengths which were obtained from the experimental flexural test are shown in Table 6. This ultimate strength was compared to the theoretical extreme in fully composite state (100% composite) to get the percentage of composite. The degree of composite of all specimens are included in Table 7 and results of ANSYS 18.2 software are shown in Table 6. Table 7 also shows the degree of composite (%), weight, and (strength/weight) ratio. The comparisons of the predictions using the analytical model that developed in this study and theoretical and experimental results from the testing described above were performed to get the conclusion of the research and the optimum parameters of it.

The highest value of actual failure load and the degree of composite action were those of specimen 03 containing LHSC with the thickness of 4.5 cm and the polyethylene foam as core material. The highest strength/weight ratio was that of specimen 06 containing LHSC with the thickness of 2.5 cm by 1.40 kN/kg. Then, that of specimen 01 containing HSC with thickness of 4.5 cm and high-density foam by 1.17 kN/kg. Also, it was noticed that specimens contain polyethylene foam were the nearest specimen which can be simulated numerically in ANSYS software and obtained the high ratio of predicted overall failure loads. In general, the upgrading grade of concrete increased significantly the total strength of slabs, the composite degree, and strength/weight ratio but the upgrading grade opposites increasing weight must be under control to observe the ratio (strength/weight). The lightweight concrete (37 MPa) and the high strength concrete (80.3 MPa) were effective in panels than light high strength concrete (70.4 MPa). That was because of very lightweight of LWC and very high strength of HSC that increased the overall (strength/weight) ratio. Generally, it was found that decreasing thickness of concrete face wythes had a positive effect on strength/weight ratio (although the ultimate loads decreased) that enhanced the performance of panels as light weight structural panels.

Failure shapes at experimental and ANSYS software are shown in Figure 13 and Figure 14. The experimental failure shape and deformation is similar to the failure shape of the numerical model especially in the 15 cm in mid-span. That proved that the numerical modeling presents specimens perfectly.

The range of results of degree of composite action degree (in form of %) of this study was almost the same range of some similar previous studies that exist in the references. The Table 8 shows the range of values of composite action in different studies.
Table 6. Results of experiments, ANSYS and theoretical extremes.

| No. | ANSYS Failure Load (kN) | Actual Failure Load (kN) | Theoretical non-Composite Load (kN) | Theoretical fully Composite Load (kN) |
|-----|-------------------------|--------------------------|------------------------------------|-------------------------------------|
| 1   | 85.32                   | 76.63                    | 31.95                              | 90.04                               |
| 2   | 81.06                   | 61.78                    | 31.64                              | 89.88                               |
| 3   | 79.78                   | 79.80                    | 31.64                              | 89.88                               |
| 4   | 65.70                   | 50.20                    | 31.64                              | 89.88                               |
| 5   | 72.10                   | 52.01                    | 28.35                              | 88.24                               |
| 6   | 47.43                   | 51.82                    | 16.45                              | 89.88                               |
| 7   | 56.68                   | 50.46                    | 24.04                              | 89.88                               |

Table 7. The degree of composite (%), weight (Kg) and (strength/weight) ratio (kN/Kg)

| No. | Degree of composite (%) | Weight (Kg) | (Strength /Weight) (kN/Kg) |
|-----|-------------------------|-------------|--------------------------|
| 1   | 85.11%                  | 65.30       | 1.17                     |
| 2   | 68.74%                  | 60.35       | 1.02                     |
| 3   | 88.79%                  | 60.31       | 1.32                     |
| 4   | 55.85%                  | 60.28       | 0.83                     |
| 5   | 58.94%                  | 47.75       | 1.09                     |
| 6   | 57.65%                  | 36.95       | 1.40                     |
| 7   | 56.14%                  | 48.65       | 1.04                     |

Table 8. The range of composite action degree comparing to previous studies

| Parameter                     | Range (%) | Reference |
|-------------------------------|-----------|-----------|
| Degree of composite action (%)| 78-86     | [3]       |
| Degree of composite action (%)| 24-91     | [5]       |
| Degree of composite action (%)| 50-80     | [6]       |
| Degree of composite action (%)| 94-100    | [10]      |
| Degree of composite action (%)| 74-84     | [12]      |
| Degree of composite action (%)| 73-89     | [15]      |
| Degree of composite action (%)| 56-89     | This study|

5.2. Discussion

Concrete Grade’s Effect:

Concrete grade was changed with different grades of concrete and cores materials to measure how it affects the ultimate strength and the degree of composite. Strength/weight ratio was calculated to measure the relative relationship between strength and weight. While comparing specimens 01 and 02, it was noticed that the ultimate strength decreased with change of HSC to LHSC by 19.33%, the degree of composite action decreased by 19.23%, and strength/weight ratio decreased by 12.8%. Furthermore, the strengths of the HSC panel and the LHSC panel were about 89.81% and 76.21% of the strengths obtained from ANSYS software respectively as shown in Figure 7 and Figure 8. When comparing specimens 01 and 05, the ultimate strength decreased with the change of HSC to LWC by 32.12%, the degree of composite action decreased by 30.74%, and strength/weight ratio decreased by 6.8%. Furthermore, the strength of the LWC panel was about 72.13% of the strength obtained from ANSYS software. When comparing specimens 2 and 5, the ultimate strength decreased by 15.8% when concrete changed from LHSC to LWC, also degree of composite action decreased by 14.2%, and by the way strength/weight ratio increased by 6.4% as shown in Figure 7 and Figure 8. The previous discussion revealed that the demand of light weight should be taken into consideration beside strength. Therefore, the smarter panels in overall (strength/weight) ratio is that of very high strength (80 MPa) or that of the very light-weight (37 MPa). In addition, that of the medium weight and strength (70 MPa), that didn’t meet the optimum parameters comparing to the other panels. That because the opposite increase in (strength/weight) ratio was not significantly observed in LHSC.
Concrete Face Wythe Thickness’s Effect

Concrete face wythe thicknesses have been changed between different three values. That parameter can judge the panels well because the decreases of the wythe thickness. When it decreases, the ultimate capacity decreases and the overall weight of the panel decreases leading to increasing (strength/weight) ratio. Changes in load capacity, composite degree, and strength/weight were observed. When comparing specimens 2 and 7, the ultimate load and the degree of composite decreased by 18.32%, but the (strength/weight) ratio increased slightly by 2% when the thickness of both wythes changed from 45 mm to 35 mm. The ANSYS results revealed that the experimental results achieved 76.21% and 89.9% from software results respectively for the panel with thicknesses of 45mm and 35mm. When comparing specimens 6 and 7, the ultimate load increased by 3%, the degree of composite increased by 3%, and strength/weight increased by 25% when the thickness changed from 35mm to 25mm as shown in Figure 9. The observed change in strength/weight ratio is subjected to the increased weight of slab due to increasing of concrete wythe. That increase was with slight change in ultimate load. When comparing specimens 2 and 6, it was observed...
that the ultimate load decreased by 15.8\% when the wythe thickness decreased from 45\text{mm} to 25\text{mm}, also composite action decreased by 16.1\% on the other side the strength/weight ratio increased by 13.3\% as shown in Figure 10. The enhanced flexural behavior was observed with the thickness of 4.5 \text{cm} and it decreases with decrease of thickness. On the other hand, the smarter panel is that of face thicknesses of 2.5 \text{cm} which achieved the highest (strength/ weight) ratio of 1.4 \text{kN/kg}.

**Figure 9. Concrete thickness effect on experimental, theoretical and ANSYS capacities**

**Figure 10. Concrete thickness effect on degree of composite and strength/weight ratio**

**Core Material Effect:**

Core material should be quite rigid to resist the shear transfer between two concrete wythes. It also has enough flexibility to transfer few forces during the test. That can be affected indirectly approaching the composite action.
When comparing specimens 2 and 3, the ultimate load and the composite degree increased by 22.5%, and strength/weight increased by 22.7% when core material was changed from HD foam to PE foam.

The panel contains PE foam achieved approximately the ANSYS result that presents 89% of theoretically fully composite state. When comparing specimens 2 and 4, the ultimate load and the composite degree decreased by 19%, and strength/weight decreased by 19% when core material was changed from HD foam to palm bark as shown in Figure 11. When comparing specimens 3 and 4, it was found that the ultimate load and composite action decreased by 37% when core material changed from PE foam to palm bark and strength/weight ratio decreased by 37.1% as shown in Figure 12. That parameter revealed that the PE foam is the best core material which provides light weight, enough flexibility, and enough rigidity to transfer slight internal forces. It was observed that PE foam has the ability to present core material in composite section in ANSYS software.

![Figure 11. Core material effect on experimental, theoretical and ANSYS capacities](image1)

![Figure 12. Core material effect on degree of composite and strength/weight ratio](image2)
Figure 13. Failure shape in ANSYS 18.2 software

Figure 14. Failure shape at experiment

Figure 15. Equivalent stress in numerical modeling

Figure 16. Equivalent elastic strain in numerical modeling
Analysis with SPSS Software:

Experimental results were analyzed with the aid of SPSS software program to get correlation and determine significantly for each parameter (concrete grade, concrete thickness, and core material). Analysis type was multiple range test (0.05) with separated parameters. Results were shown in Table 9.

Table 9. SPSS software results

| Parameter       | NO. of specimens | Correlation | Level       |
|-----------------|------------------|-------------|-------------|
| Concrete Grade  | 1, 2, 5          | Significant | 0.05 (2-tailed) |
| Concrete Thickness | 2, 6, 7       | Significant | 0.01 (2-tailed)  |
| Core Material   | 2, 3, 4          | Significant | 0.01 (2-tailed)  |

6. Conclusions

Based on the results of this experimental investigation under tidal environment, the following conclusions are drawn as following:

- High strength concrete (HSC) and lightweight concrete (LWC) are very successful in the production face wythes of precast lightweight sandwich panels.
- Although the loading capacity of panels increased with the increase of face wythes thicknesses, the overall weight of panels increases.
- The optimum face wythe thickness is the that of 2.5 cm which has high (strength/weight) ratio of 1.4 kN/kg although it has relatively low ultimate capacity.
- Adding polyethylene foam as a core material results positive effect on structural sandwich panel. It results high (strength/weight) ratio of 1.32 kN/kg because of its relative lightweight and flexibility.
- The polyethylene foam was the nearest core material which presented its real role in modeling of ANSYS software because of it enough rigid and flexible to transfer internal forces in panel.
- In general, upgrading grade of concrete increased significantly the total strength of slabs, the composite degree, and strength/weight ratio. On the other hand, when upgrading grade opposite increasing weight must be under control to observe the ratio (strength/weight). The lightweight concrete (37 MPa) and the high strength concrete (80.3 MPa) were effective in panels than light high strength concrete (70.4 MPa). That was because of very lightweight of LWC and very high strength of HSC which increased the overall (strength/weight) ratio.
- Generally, it was found that the optimum thickness is 2.5 cm, the optimum core material is polyethylene foam, and the optimum type of concrete is high strength concrete (80 MPa and density = 2660 kg/m³).
- Analysis with SPSS software program showed that all parameter correlations (concrete grade, concrete thicknesses and core materials) were significant.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. Nomenclature

SPSP = Structural precast sandwich panel;  
HSC = High strength concrete;  
LHSC = Light high strength concrete;  
LWC = Light weight concrete;  
HD foam = High density foam;  
PE foam = Polyethylene foam;  
Palm bark = the bark of palm tree;  
Steel Ø 12 mm = Steel reinforcement with diameter of 12 mm.
9. References

[1] Choi, Wonchang, Seok-Joon Jang, and Hyun-Do Yun. “Design Properties of Insulated Precast Concrete Sandwich Panels with Composite Shear Connectors.” Composites Part B: Engineering 157 (January 2019): 36–42. doi:10.1016/j.compositesb.2018.08.081.

[2] Lee, Ji-Hyung, Sung-Hoon Kang, Yu-Jin Ha, and Sung-Gul Hong. “Structural Behavior of Durable Composite Sandwich Panels with High Performance Expanded Polystyrene Concrete.” International Journal of Concrete Structures and Materials 12, no. 1 (February 22, 2018). doi:10.1186/s40069-018-0255-6.

[3] Insub Choi, JunHee Kim, and Young-Chan You, “Effect of cyclic loading on composite behavior of insulated concrete sandwich wall panels with GFRP shear connectors.” Composites Part B: Engineering, Volume 96, (July 1, 2016): 7–19. doi:10.1016/j.compositesb.2016.04.030.

[4] Mohamed Mahdy, Ahmed Reheem and P. Speare. “Mechanical properties of heavyweight, high strength concrete.” Proceedings of the annual Conference of the Canadian Society for Civil Engineering, Montréal, Canada, (June 5, 2002):5-8.

[5] Douglas Tomlinson and Amir Fam, “Flexural behavior of precast concrete sandwich wall panels with basalt FRP and steel reinforcement.” PCI Journal, vol. 60, no. 6, (November 1, 2015): 51–71. doi:10.15554/pci.j.11012015.51.71.

[6] Douglas Tomlinson and Amir Fam, “Experimental investigation of precast insulated sandwich panels with glass fiber-reinforced polymer shear connectors.” ACI Structural Journal, vol. 111, no. 3, (January 5, 2014): 595-606. doi:10.14359/51686621.

[7] J. Daniel Ronald Joseph, J. Prabakar, and P. Alagusundaramoorthy, “Flexural behavior of precast concrete sandwich panels under different loading conditions such as punching and bending.” Alexandria Engineering Journal, vol. 57, no. 1, (March 1, 2018): 309–320. doi:10.1016/j.aeje.2016.11.016.

[8] Wahid Ferdous, Allan Manalo, Thiru Aravinthan, and Amir Fam, “Flexural and shear behaviour of layered sandwich beams.” Construction and Building Materials, vol. 173, (June 10, 2018): 429–442. doi:10.1016/j.conbuildmat.2018.04.068.

[9] Jauhar Fajrin, Yan Zhuge, Hao Wang and Frank Bullen, “Experimental and Theoretical Deflections of Hybrid Composite Sandwich Panel under Four-point Bending Load.” Civil Engineering Dimension, vol. 19, no. 1, (March 1, 2017): 29–35. doi:10.9744/ced.19.1.29-35.

[10] J. Daniel Ronald Joseph, J. Prabakar, and P. Alagusundaramoorthy, “Precast concrete sandwich one-way slabs under flexural loading.” Engineering Structures, vol. 138, (May 1, 2017): 447–457. doi:10.1016/j.engstruct.2017.02.033.

[11] Won-Hee Kang and JunHee Kim, “Reliability-based flexural design models for concrete sandwich wall panels with continuous GFRP shear connectors.” Composites Part B: Engineering, vol. 89, (March 15, 2016): 340–351. doi:10.1016/j.compositesb.2015.11.040.

[12] Y.H. Mugahed Amran, Raizal S.M.Rashid, Farzad Hejazi, Nor Azizi Safiee, and A.A. Abang Ali, “Response of precast foamed concrete sandwich panels to flexural loading.” Journal of Building Engineering, vol. 7, (September 1, 2016): 143–158. doi:10.1016/j.jobe.2016.06.006.

[13] Mohamed Mahdy, “Structural Lightweight Concrete Using Cured Leca.”, International Journal of Engineering and Innovative Technology (IJET), Volume 5, Issue 9, (March 1, 2016): 25-31. doi:10.17605/osf.io/mcvb3.

[14] Mohamed Mahdy, Mahmoud Imam, Ahmed Tahwia, and Osama Yousef, “Study on Heat Transfer of Sandwich Panels.” Proceedings of the 7th International Engineering Conference, Faculty of Engineering, Mansoura University, Egypt, volume 1, (March 23, 2010).

[15] A. Benayoune, A.A. AbdulSamad, D.N.Trikha, A.A. Abang Ali, and S.H.M. Ellinna, “Flexural behaviour of pre-cast concrete sandwich composite panel - Experimental and theoretical investigations.” Construction and Building Materials, vol. 22, no. 4, (April 1, 2008): 580–592. doi:10.1016/j.conbuildmat.2006.11.023.

[16] Mohamed Mahdy, Mahmoud Imam, Ahmed Tahwia, and Osama Yousef, “Experimental Study of Structural Sandwich Members.”, Housing & Building National Research Journal (HBRC), Vol 7, No 3, (December 1, 2011): 48-58.

[17] R., Balamuralikrishnan, and Saravanan J. “Finite Element Analysis of Beam – Column Joints Reinforced with GFRP Reinforcements.” Civil Engineering Journal 5, no. 12 (December 1, 2019): 2708–2726. doi:10.28991/cej-2019-03091443.
Analyzing Engineering-Related Delays Using Quality Function Deployment in Construction Projects

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Abstract
This paper presents a methodology for analyzing engineering-related delays in construction projects using Quality Function Deployment (QFD). The steps of the QFD technique are combined in the quality and control policy. A reference matrix based on the literature review is constructed with engineering delays and a survey of all parties involved in construction projects. The QFD matrix aids in identifying the most significant reasons for delays and claims in the construction projects. For the identified reasons, solutions have been developed to limit or reduce them. The mean sources of construction delays include engineering, construction, financial/economic, management/administrative, and force majeure. This paper presents a knowledge-based QFD technique dedicated to engineering-related delays. Three categories of Engineering-related delays are considered in the proposed system. These categories are 1) design development, 2) workshop drawings, and 3) project party’s changes delays. The knowledge of the QFD matrix is acquired from literature, Federation International des Ingenious - Conselis (FIDIC) contract forms, domain experts, as well as a questionnaire survey. Three classes of participants (i.e., consultants, contractors, and Employers) have been approached to get their feedback on the cases of engineering-related delays. The proposed approach helps to limit or reduce delays in construction projects caused by the engineer. Accordingly, it was concluded to the most important reasons that led to the delay of construction projects related to the engineer, using QFD.

Keywords: Engineering Delays; Construction Projects; Quality Function Deployment; Questionnaire Survey; FIDIC.

1. Introduction

The construction industry has different characteristics that may lead to delays, which might lead to disputes between the various parties of the project. The flexibility of owners to make changes during the execution phase, the distribution of risks between owners and contractors, and the degree of owner’s involvement in the project control during construction time may vary from a procurement strategy to another. The procurement strategy would be more concerned about defining the appropriate project delivery method and selecting the best contract type that suits the project environment and objectives. Delays and claims are common due to the increasing complexity of the construction process. Owners used to transfer the major risks to contractors. These risks include; inflation, accidents, low labor productivity, adverse weather, shortage of materials and skilled labor, and unforeseen site conditions. Thus, construction contracts are becoming more complex. Delays and claims have become a repetitive phenomenon in the construction industry. Such a phenomenon, if not managed efficiently, would hinder the success of many construction projects, and thus slow down the wheel of development.

This research proposed the use of QFD technique as a preventive procedure to reduce engineering delays in construction projects. The use of the QFD matrix improves the quality and reliability of engineer-related work and
thus minimizes delays in construction projects that may lead to claims or disputes. The rest of the paper is organized as follows: Section 2 includes the literature review. Then the proposed QFD Methodology is illustrated in detail in Section 3. Section 4 presents the data collection and detailed analysis of the engineering-related key delays. Then the evaluation of the findings is described in Section 5. Finally, the results of this study are concluded in Section 6.

2. Literature Review

Quality function deployment consists of four stages, which are summarized according to previous studies. Product Planning is the first stage in which the user’s requirement converted into design specifications. Then, these specifications are prioritized, and the design target values are finalized. The essential characteristics of the product are then published in the next phase of QFD. This matrix is called House of Quality [1]. House of Quality (HOQ) is the first and most significant matrix of QFD explained in the following steps [2]:

- Determine the requirements and needs of users, and then put them in the first column of the matrix [3].
- Assign the priority value next to each of the requirements (degree of importance) by using the Likert scale after making a survey of the users and place those values in a column next to the needs [4].
- The designing team determines the design specifications, which correspond to users’ needs. This is considered a significant step in the translation process, as it requires a lot of research and professional expertise in various aspects of designing in order to reach the product characteristics [3].
- Competitive Analysis: set by the user in order to determine which of the designing team has fulfilled needs [5].
- The relationship between design specifications and the requirements of the user (Relationship Matrix) (Figure 3-5): Determined by the designing team where the relationship between the requirements of users and the design specifications is described in the numerical value of (0 =No correlation, 1 =Weak correlation, 3 = Medium correlation, 9 = Strong correlation) [6]. Such evaluation is driven by personal experience, user survey results, or data from statistical studies. (Figure 1)
- Correlation among design specifications: (Correlation Matrix) (Figure 2): Designing team determines how each of the design specifications affects the other specifications. The correlation is expressed as a strong positive correlation or a negative correlation relationship. This matrix is utilized less frequently in quality houses. However, it provides great help for designers during the next phase of QFD [7].
- Determine the Importance Weight to specifications of design user requirements in the previous matrix are replaced by the design specifications while the design specifications are replaced by design components [4]: This equals the sum of multiplying the degree of importance to a need by the value of the relationship between that need and the corresponding design specification [8].
- Determine the Relative Weight of the design specification: To evaluate Relative Weight to each of the design specifications, each Importance Weight is divided by total Importance Weight to all specifications then multiplied by 100.
Then the Product design stage requires designing team to come up with creative and innovative ideas. The concepts of design re-established in order to achieve target values on a priority basis. Such phase involves the following steps [9]:

- User requirements in the previous matrix are replaced by design specifications, while design specifications are replaced by design components [4].
- The degree of importance of each of the design specifications is are calculated according to the Relative Weight discovered in the first matrix.
- The correlation between design specifications and the design components in the midsection of the matrix is determined by designers [10].
- The Importance weight of each of the design components equals to the sum of multiplying Degree of Importance of any design specifications by Correlation Value of the related Design Components.
- The Relative Weight to each of the Design Components is determined by dividing each Importance Weight of the component by the total Importance Weights to all components then multiplied by 100.

After that, the Process Planning phase in which we identify the work required to prepare each of the components by characterizing the required processes to accomplish our task [11]. Then the Process Control phase where critical control measures are set in order to prevent failure in coordinating with the department of quality assurance to define performance indicators to monitor the production process [12].

2.1. Quality Function Deployment

Quality Function Deployment (QFD) is defined as a method for developing the design quality which aims at satisfying the consumer and then translating the demand of consumer into design targets and major quality assurance points to be used throughout the production phase. QFD can be seen as a process in which the consumer’s voice is valued to carry through the whole process of production and services. QFD was invented in Japan by Yoji Akao in 1966 but was first implemented in the Mitsubishi’s Kobe shipyard in 1972, possibly out of the teaching of Deming [13]. Then, later it was adopted and developed by other Japanese companies, notably Toyota and its suppliers. The long-term viability of an organization mainly depends on how effectively the organization utilizes its resources to satisfy its stakeholders. For the organizations operating in the construction industry, one of the most privileged stakeholders is the clients (end-users or customers depending on the project type; therefore, in the rest of this research, client, customer, and end-user will be used interchangeably). Satisfying their needs and expectations is of the uttermost importance for the companies because the quality is in the eye of the beholder, and whatever they demand and expect from a product/project defines the quality characteristics of an entity. The unique nature of the industry necessitates the understanding of client needs and expectations for each project carefully for increasing their satisfaction level. Over the past decades, quality has been a differentiating factor within the construction industry. It has been demonstrated that despite the constraints on quality differentiation efforts (like project budget, rules, and regulations, etc.), many companies are competing using quality differentiation strategy and sustaining their competitiveness in the long run [8].

Achievement of client satisfaction necessitates the management of quality systematically, which further requires the utilization of quality tools and techniques for this purpose. Quality function deployment (QFD) is one of these techniques to deal with customer needs and expectations more systematically for achieving the most significant objective of a construction company, satisfaction of clients. QFD is broadly total quality management (TQM) implementation technique requiring a clear assessment of client/end-user expectations apart from the basic needs of a project to convert them into design targets. It is worth noting that Quality Function Deployment (QFD) allows the consideration of the "voice of the customer" along the service development path to market entry [14]. A structured approach of designing, by translating user's requirements into design characteristics during each phase of the product development process [15]. A way to ensure the quality of design when the product is in the design study phase [11]. Methodology to focus on various dimensions of quality during the product design process [7].

2.2. QFD-TECHNIQUE

The QFD technique is based on the analysis of the clients’ requirements, which normally are expressed in qualitative terms, such as: “easy to use,” “safe,” “comfortable,” or “luxurious.”
To develop a service, it is necessary to “translate” these fuzzy requirements into quantitative service design requirements; QFD makes this translation possible [16]. Services are not developed as a whole; instead, these are developed through the integration of different components. The component features are what provide the functionality that, in turn, satisfy client requirements. The firm organization is another factor that affects service development. Unfortunately, the importance of the service development process is not known by all the employees. For this reason, the establishment of an appropriate communication system is particularly important. This system must keep the meaning of the clients’ requirements during the development process [14].

3. QFD Methodology

In this paper, the methodology as follows:

- Develop the customers’ requirements list. This study the Engineering-related delays (referred to as the voice of customers or VOC) [17]. It summarizes the Major Categories of Delays and Causes Tables (1 to 8).

- Rank the customers’ requirements list (Engineering-related delays). Each customer requirement will be rated according to the causes of the Engineering-related delay (usually, these ratings are assessed based on focus group sessions). The following importance weights are used: 3, 6, and 9 Tables (9 to 13).

- Use quantifiable measures the Engineering-related delays’ requirements.

- Define measurement units for technical requirements.

- Identify whether technical requirements correlate with each other. This can be defined in the triangular rooftop matrix (Figure.2). However, it is applicable to assume independence between technical requirements where this part can be dropped.

- Define the correlation between Engineering-related delays and technical requirements by assigning a weighting factor (weak = 1; moderate = 3; strong = 9) in the intersection of each row (Engineering-related delays) with each column (technical requirements). The following symbols are used: “Ω = weak,” “Ο = moderate,” and “Δ = strong.”

- Determine the relative importance of each technical requirement. For each technical requirement column, the weight rating (1, 2, or 3 of Step 6) is multiplied by the prioritization rating (determined in Step 2) for each of the Engineering-related delays. The sum of each column is written at the bottom of the column. Eldin and Hikle (2003) [1] defined the rest of the steps (Step 8–11) as follows: evaluate the current competition, determine benchmarks, determine target values, and evaluate new related delays. In this study, the findings of the previous steps (Steps 1–7) are used to reduce the Engineering-related delays on the construction projects. The rest of the steps were modified to fit the purpose of this work, as follows:

- Evaluate the current practice of each technical requirement. The technical requirements will be assessed on a Likert scale (ranging from 1 to 5), in which 5 is excellent, 3 is good, 1 is weak.

- Calculate the weights of each technical requirement as the ratio of the column sum (found in Step 7) over the total sum of the technical requirements that belong to its attribute (attribute sum).

- Evaluate the attributes. The weight of the technical requirement (TR) (found in Step 9) and the Likert scale evaluation (found in Step 8) will be used to define the attribute weighted average score (AWAS), as follows:

\[
AWAS_{attribute} = \sum (TR \text{ weight } \times \text{ likert scale})
\]  

(1)

- Determine the performance level (excellent, satisfactory, and deficient) for the 8 attributes according to AWAS [excellent (4 ≤ AWAS ≤ 5); satisfactory (3 ≤ AWAS < 4); deficient (1 ≤ AWAS < 3)].
3.1. Preparing Tender Documents (Technical Requirements)

Quality function deployment defines technical requirements as elements needed to deliver a product or a service. In this paper, TR is used in a broader sense to include managerial and planning requirements. Technical requirements are organized in the paper at three levels: phases, attributes, and detailed requirements. At the first level, two (Engineering-related delays) ERD phases are defined: ERD technical written documents, and engineering drawings. However, the definition of ERD phases differs among authors [18, 19]. Also, this research specifies 5 ERD attributes (second level) cascaded down into 36 detailed technical requirements (third level). Table 1 shows the ERD TR hierarchy. The following details are based on the literature survey.

Table 1. Engineering design of the project and preparing tender documents, Technical Requirements Summary

| Hierarchy | Engineering design of the project and preparing tender documents |
|-----------|---------------------------------------------------------------|
| 2 phases  | Requirements for technical written documents                 |
| Attributes| Requirements for special technical specifications and writing |
|           | Requirements of measurement units approved in the specifications | Requirements related to the description of implementation technology and safety requirements | Requirements Price and estimation | Requirements for the Bill of Quantities | Requirements Contract | Requirements for engineering drawings. |
|           | 8                                                             | 3                                                             | 6                                                             | 4                                                             | 4                                                             | 2                                                             | 1                                                             | 8                                                             | 28                                                             | TR 36 |

3.1.1. Requirements for Technical Written Documents

The complete Contract specifications consist of an assembly of appropriate standard and one-time-use specifications supplemented by lists and descriptions of items of work and construction details. What design errors: the study errors committed by the engineer during the preparation of any document of competition (technical documents and drawings) of the project.

a) Requirements for special technical specifications and writing

1. Requirements Specification.
Table 2. Requirements technical written documents (Specification)

| Requirements (Technical written documents) | Index | Wight | Detailed technical requirements |
|-------------------------------------------|-------|-------|-------------------------------|
| Special technical specifications and writing | TR1   | 3     | Materials accurate description and implementation methods |
|                                           | TR2   | 3     | Specifications clarity and non-generalization |
|                                           | TR3   | 3     | An exact description of certain characteristics execution |
|                                           | TR4   | 3     | Possibility of applying the practical specifications of the project |
|                                           | TR5   | 3     | Clarify the measurement methods used and conform to what is stated in other parts of the other project documents of drawings, drawings, bill of quantities, etc. |
|                                           | TR6   | 3     | Characterization of test methods for construction materials |
|                                           | TR7   | 3     | Avoid repetition of Specific work descriptions in two different formats or specifications conflict with other contract documents such as drawings |
|                                           | TR8   | 3     | Avoid using unknown standard specifications that may lead to misunderstanding |

2. Requirements of measurement units approved in the specifications.

Table 3. Requirements technical written documents (Measurement units approved in the specifications)

| Requirements (Technical written documents) | Index | Wight | Detailed technical requirements |
|-------------------------------------------|-------|-------|-------------------------------|
| Special technical specifications and writing | TR9   | 3     | Avoid contrast and difference in units of measurement considered in different parts of the study |
|                                           | TR10  | 3     | Clarify what includes the implementation of the unit of measure of work |
|                                           | TR11  | 3     | Selection of the appropriate measurement unit |

3. Requirements Printing, Drafting, and writing.

Table 4. Requirements technical written documents (Printing.)

| Requirements (Technical written documents) | Index | Wight | Detailed technical requirements |
|-------------------------------------------|-------|-------|-------------------------------|
| Special technical specifications and writing | TR12  | 3     | Review and check the technical conditions after the last printing, especially concerning methods and units of measurement and punctuation marks, etc. |
|                                           | TR13  | 2     | Divide project work into sections, chapters, and paragraphs properly fit the work received in the project |
|                                           | TR14  | 2     | Use punctuation correctly |
|                                           | TR15  | 2     | Avoid using long and weak sentences |
|                                           | TR16  | 2     | Use understandable and known expressions |
|                                           | TR17  | 2     | Avoid using general words |

4. Requirements were related to the description of implementation technology and safety requirements.

Table 5. Requirements technical written documents (description of implementation technology and safety)

| Requirements (Technical written documents) | Index | Wight | Detailed technical requirements |
|-------------------------------------------|-------|-------|-------------------------------|
| Special technical specifications and writing | TR18  | 2     | Describe construction methods details |
|                                           | TR19  | 2     | Consider the execution ability method contained in the technical terms |
|                                           | TR20  | 2     | Description of procedures security and public safety |
|                                           | TR21  | 2     | Statement of implementation method clearly or in a manner that does not conflict with the rest of the tender documents |

b) Requirements Price and estimation

Table 6. Requirements technical written documents (Price and estimation)

| Requirements (Technical written documents) | Index | Wight | Detailed technical requirements |
|-------------------------------------------|-------|-------|-------------------------------|
| Price and estimation | TR22  | 2     | Approve the prices received with the required specifications |
|                                           | TR23  | 2     | Adequate and detailed price data |
|                                           | TR24  | 3     | Avoid omission of the analysis or estimate of the price of the material or work required to implement an item |
|                                           | TR25  | 3     | Avoid contrast and difference between the measurement unit used in pricing in both the Bill of Quantities and the Price Table or the specifications |
c) Increments for the Bill of Quantities

Table 7. Requirements technical written documents (Bill of Quantities)

| Requirements (Technical written documents) | Index | Wight | Detailed technical requirements |
|--------------------------------------------|-------|-------|--------------------------------|
| Bill of Quantities                         | TR26  | 3     | Estimate the correct quantities of project works and avoid the exceeding, more than the specified percentage |
|                                            | TR27  | 3     | Calculation of quantities based on detailed and final plans |

d) Requirements Contract

Table 8. Requirements technical written documents (Contract)

| Requirements (Technical written documents) | Index | Wight | Detailed technical requirements |
|--------------------------------------------|-------|-------|--------------------------------|
| Contract                                    | TR28  | 3     | Avoid the difference between the clauses and terms of the contract and the general conditions |

3.1.2. Requirements for Engineering Drawings

Table 9. Requirements for engineering drawings.

| Index | Wight | Detailed technical requirements |
|-------|-------|---------------------------------|
| TR29  | 3     | Design conforms to the wishes of the project owner or model design criteria |
| TR30  | 3     | The design conforms to code requirements |
| TR31  | 3     | Construction calculations match (Avoiding calculation errors) |
| TR32  | 3     | Avoid differences and inconsistencies among the different drawings (coordination flaw between different drawings: architectural, structural, civil, etc.) |
| TR33  | 3     | Operability/constructability problems • Lack of clarity on the construction and implementation mechanism, especially in non-recurrent special construction works, and in reinforcement and maintenance projects |
| TR34  | 3     | Scale or dimensional errors |
| TR35  | 3     | The need for adequate architectural and construction details to complete the work as required |
| TR36  | 3     | Compliance with standards and formalizations of drawing and avoiding errors |

3.2. Engineering-Related Delays (Customers’ Requirements)

Table 10. Design development delays

| The main source of delay | Index | Importance | Categories of Delays and Causes |
|-------------------------|-------|------------|---------------------------------|
| Design development delays | ERD1  | Strong (weight = 9) | Delay in receiving the design criteria that are needed to start the design process |
|                         | ERD2  | Strong (weight = 9) | Mistakes/changes in the design criteria provided by the employer |
|                         | ERD3  | Strong (weight = 9) | Delay in responding to contractor’s queries |
|                         | ERD4  | Strong (weight = 9) | Delay in the approval stage |
|                         | ERD5  | Strong (weight = 9) | Delay in the design process due to lack of resources, experience, management, etc. |
|                         | ERD6  | Strong (weight = 9) | Delay due to mistakes in the generated design documents |
|                         | ERD7  | Strong (weight = 9) | Delay due to unforeseen conditions in design development |

Table 11. Workshop drawing submission delays

| The main source of delay | Index | Importance | Categories of Delays and Causes |
|-------------------------|-------|------------|---------------------------------|
| Workshop drawing submission delays | ERD8  | Strong (weight = 9) | Delay in receiving design documents that are needed to start the preparation of the workshop drawings process |
|                         | ERD9  | Strong (weight = 9) | Mistakes/changes in the design documents provided by the employer |
|                         | ERD10 | Strong (weight = 9) | Delay in responding to contractor’s queries |
|                         | ERD11 | Strong (weight = 9) | Delay in the preparation process due to lack of resources, experience, management, etc. |
|                         | ERD12 | Strong (weight = 9) | Delay due to unforeseen conditions in shop drawings submission |
Table 12. Workshop drawing approval delays

| The main source of delay | Index | Importance | Categories of Delays and Causes |
|-------------------------|-------|------------|---------------------------------|
| Workshop drawing approval delays | ERD13 | Strong (weight = 9) | Delay in receiving the needed information to start the review of the workshop drawings process |
|                         | ERD14 | Strong (weight = 9) | Mistakes/changes in the generated shop drawings |
|                         | ERD15 | Strong (weight = 9) | Delay in responding to employer’s queries |
|                         | ERD16 | Strong (weight = 9) | Delay in the approval process due to lack of resources, experience, management, etc. |
|                         | ERD17 | Strong (weight = 9) | Delay due to unforeseen conditions in the approval stage |

Table 13. Project parties’ changes delays

| The main source of delay | Index | Importance | Categories of Delays and Causes |
|-------------------------|-------|------------|---------------------------------|
| Project parties’ changes delays | ERD18 | Strong (weight = 9) | Changes due to mistakes/contradiction and/or constructability problems in the generated design documents |
|                         | ERD19 | Strong (weight = 9) | Changes in construction procedure due to unforeseen site condition (s) |
|                         | ERD20 | Strong (weight = 9) | Changes in construction procedure due to soil investigation problem (s) |
|                         | ERD21 | Strong (weight = 9) | Changes in specifications to save time and/or cost |
|                         | ERD22 | Strong (weight = 9) | Changes in specifications due to unavailability of materials |

4. Data Collection and Analysis

Detailed analysis of the engineering-related key delays is presented as a summary to the knowledge, which had been extracted from the Studies and previous research and also experts in this field. Also, a questionnaire survey had been carried out by the present research writer to ensure the accuracy of the stated summary. Both stages (extracting the knowledge from the previous research and the questionnaire survey) are representing the most important phase in achieving the objectives of the present study since the outcome of these stages represents the core of a QFD-methodology for assessing the delays caused through engineering-related attributes. Also, a detailed analysis of the Requirements for the Engineering design of the project and preparing tender documents, which had been extracted from the Studies and previous research and also experts in this field.

Table 14. Distributed questionnaire sample

| Party       | No. of Questionnaires |
|-------------|-----------------------|
| Owners      | 23                    |
| Consultants | 23                    |
| Contractors | 23                    |
| Total       | 69                    |

4.1. Questionnaire Contents

The data included in the questionnaire is divided into four parts. These four parts are:

Part 1: Personal information
Part 2: Organizational information
Part 3: Engineering design of the project and preparing tender documents
Part 4: Engineering-Related Delays (Major Categories of Delays and Causes)

The stakeholders’ requirements will be evaluated according to their importance for the ERD program. The researchers suggest three levels of importance with different weights. The levels include Strong (9), Moderate (3), and Weak (1) importance.

Tables 10 to 13 shows four main sources of delay. Engineering-Related Delays with 22 requirements.
5. Evaluation

The findings of the focus groups sessions are summarized as follows:

1. The TRS (Tables 2 to 9) are used as column headings, and the Engineering-Related Delays (ERD) (Table 10 to 13) are used as row headings.

2. At the intersection for each TR (column) and ERD (row), the correlation is evaluated according to three weights (strong = 9; moderate = 3; and weak = 1). The intersection is filled with “⊕,” “〇,” and “△” as shown in Table 15.

| Correlation level | Symbol | weight |
|-------------------|--------|--------|
| Strong            | ⊕      | 9      |
| Moderate          | 〇      | 3      |
| weak              | △      | 1      |

3. For each TR (column), the total weighted correlation is calculated as the sum of products of stakeholder importance and its correlation weight.

\[ TR_j = \sum_{n=1}^{N} (\text{Engineering Related Delays (ERD)} \times \text{correlation weight})_j, \]

\[ J = 1, 2, \ldots, 36 \quad (2) \]

4. The attribute’s sum will be the added sum of its consisted TRs

\[ \text{Attribute sum} = \sum (\text{TR sum}) \quad (3) \]

5. The TR weights will be the ratio of TR sum over its attribute sum

\[ TR \ weight = \frac{TR \ sum}{Attribute \ sum} \quad (4) \]

For each attribute, the total of its consisting TR weights will be 1. Tables 15(A) to 15(C) show the QFD matrices.

6. The TRs is evaluated according to the Likert scale (ranging from 1 to 5); the research assessed the 32 technical requirements, where 5 is excellent, 4 is very good, 3 is good, 2 is fair, and 1 is poor.

7. The AWAS is calculated for 10 attributes;

\[ AWAS_{attribute} = \sum (\text{TR weight + likert scale}) \quad (5) \]

8. A performance level (excellent, satisfactory, or deficient) is determined for the 10 attributes based on the AWAS, as described in Table 16.

| AWAS range | Evaluation level |
|------------|------------------|
| 4 \leq \text{AWAS} \leq 5 | 1 (excellent) |
| 3 \leq \text{AWAS} < 4 | 2 (satisfactory) |
| 1 \leq \text{AWAS} < 3 | 3 (deficient) |

Table 18 shows the evaluation output of AWAS and the performance levels of the attributes for the being studied.
### Table 17(A). Correlation matrix

| ERD main source of delay | Categories of Delays and Causes | Importance |
|-------------------------|---------------------------------|------------|
|                         |                                 | TR1 | TR2 | TR3 | TR4 | TR5 | TR6 | TR7 | TR8 | TR9 | TR10 | TR11 |
| 1                       | Delay in receiving the design criteria that are needed to start the design process. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 2                       | Mistakes/changes in the design criteria provided by the employer. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 3                       | Delay in responding to contractor’s queries. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 4                       | Delay in approval stage. | O   | O   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 5                       | Delay in the design process due to lack of resources, experience, management, etc. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 6                       | Delay due to mistakes in the generated design documents. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 7                       | Delay due to unforeseen conditions in design development. | O   | O   | O   | O   | O   | O   | O   | O   | O   | O    | O    |
| 8                       | Delay in receiving design documents that are needed to start the preparation of the workshop drawings process. | O   | O   | O   | O   | O   | O   | O   | O   | O   | O    | O    |
| 9                       | Mistakes/changes in the design documents provided by the employer. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 10                      | Delay in responding to contractor’s queries. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 11                      | Delay in the preparation process due to lack of resources, experience, management, etc. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 12                      | Delay due to unforeseen conditions in shop drawings submission. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 13                      | Delay in receiving the needed information to start the review of the work shop drawings process. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 14                      | Mistakes/changes in the generated shop drawings | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 15                      | Delay in responding to employer’s queries. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 16                      | Delay in the approval process due to lack of resources, experience, management, etc. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 17                      | Delay due to unforeseen conditions in approval stage | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 18                      | Changes due to mistakes/contradiction and/or constructability problems in the generated design documents. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 19                      | Changes in construction procedure due to unforeseen site condition(s) | O   | O   | O   | O   | O   | O   | O   | O   | O   | O    | O    |
| 20                      | Changes in construction procedure due to soil investigation problem(s). | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 21                      | Changes in specifications in order to save time and/or cost. | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ   | Δ    | Δ    |
| 22                      | Changes in specifications due to unavailability of materials | O   | O   | O   | O   | O   | O   | O   | O   | O   | O    | O    |
Table 17(B). Correlation matrix

| Eradication source of delay | Categories of Delays and Causes | Importance | Technical Requirements |
|----------------------------|---------------------------------|------------|------------------------|
|                            |                                 |            | Requirements for technical written documents |
|                            |                                 |            | Requirements for special technical specifications and writing |
|                            |                                 |            | Requirements Printing |
|                            |                                 |            | Requirements related to the description of implementation technology and safety requirements |
| 1                          | Delay in receiving the design criteria that are needed to start the design process. | 9          | TR12 TR13 TR14 TR15 TR16 TR17 TR18 TR19 TR20 TR21 |
| 2                          | Mistakes/changes in the design criteria provided by the employer. | 9          | O        O  O  O  O  O  O  O  O  O |
| 3                          | Delay in responding to contractor’s queries. | 9          | O        O  O  O  O  O  O  O  O  O |
| 4                          | Delay in approval stage. | 9          | D        D  D  D  D  D  D  D  O  O |
| 5                          | Delay in the design process due to lack of resources, experience, management, etc. | 9          | D        D  D  D  D  O  D  D  O  O |
| 6                          | Delay due to mistakes in the generated design documents. | 9          | D        D  D  D  D  O  D  D  O  O |
| 7                          | Delay due to unforeseen conditions in design development. | 9          | D        D  D  D  D  O  D  D  O  O |
| 8                          | Delay in receiving design documents that are needed to start the preparation of the workshop drawings process. | 9          | D        D  D  D  D  O  D  D  O  O |
| 9                          | Mistakes/changes in the design documents provided by the employer. | 9          | D        D  D  D  D  O  D  D  O  O |
| 10                         | Delay in responding to contractor’s queries. | 9          | D        D  D  D  D  O  D  D  O  O |
| 11                         | Delay in the preparation process due to lack of resources, experience, management, etc. | 9          | D        D  D  D  D  O  D  D  O  O |
| 12                         | Delay due to unforeseen conditions in shop drawings submission. | 9          | D        D  D  D  D  O  D  D  O  O |
| 13                         | Delay in receiving the needed information to start the review of the work shop drawings process. | 9          | D        D  D  D  D  O  D  D  O  O |
| 14                         | Mistakes/changes in the generated shop drawings. | 9          | D        D  D  D  D  O  D  D  O  O |
| 15                         | Delay in responding to employer’s queries. | 9          | D        D  D  D  D  O  D  D  O  O |
| 16                         | Delay in the approval process due to lack of resources, experience, management, etc | 9          | D        D  D  D  D  O  D  D  O  O |
| 17                         | Delay due to unforeseen conditions in approval stage. | 9          | D        D  D  D  D  O  D  D  O  O |
| 18                         | Changes due to mistakes/contradiction and/or constructability problems in the generated design documents. | 9          | D        D  D  D  D  O  D  D  O  O |
| 19                         | Changes in construction procedure due to unforeseen site condition(s) | 9          | D        D  D  D  D  O  D  D  O  O |
| 20                         | Changes in construction procedure due to soil investigation problem(s). | 9          | D        D  D  D  D  O  D  D  O  O |
| 21                         | Changes in specifications in order to save time and/or cost. | 9          | D        D  D  D  D  O  D  D  O  O |
| 22                         | Changes in specifications due to unavailability of materials | 9          | D        D  D  D  D  O  D  D  O  O |

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### Table 17(C). Correlation matrix

| Index | Categories of Delays and Causes                                                                 | Importance | Technical Requirements                                                                 |
|-------|-----------------------------------------------------------------------------------------------|------------|---------------------------------------------------------------------------------------|
|       |                                                                                               |            | Requirements for technical written documents | Requirements for special technical specifications and writing | Requirements Price and estimation | Requirements for the Bill of Quantities | Requirements Contract |
|       |                                                                                               |            | TR22 | TR23 | TR24 | TR25 | TR26 | TR27 | TR28 |
| 1     | Delay in receiving the design criteria that are needed to start the design process.             | 9          | ☐    | ☐    | ☐    | ☐    | Δ    | Δ    | ☐    |
| 2     | Mistakes/changes in the design criteria provided by the employer.                               | 9          | ☐    | ☐    | ☐    | ☐    | O    | O    | ☐    |
| 3     | Delay in responding to contractor’s queries.                                                    | 9          | ☐    | ☐    | ☐    | ☐    | O    | O    | ☐    |
| 4     | Delay in approval stage.                                                                         | 9          | ☐    | ☐    | ☐    | ☐    | O    | O    | ☐    |
| 5     | Delay in the design process due to lack of resources, experience, management, etc.              | 9          | O    | O    | O    | O    | O    | O    | ☐    |
| 6     | Delay due to mistakes in the generated design documents.                                        | 9          | ☐    | ☐    | ☐    | ☐    | O    | Δ    | ☐    |
| 7     | Delay due to unforeseen conditions in design development.                                       | 9          | Δ    | Δ    | Δ    | Δ    | Δ    | Δ    | Δ    |
| 8     | Delay in receiving design documents that are needed to start the preparation of the workshop drawings process. | 9          | Δ    | Δ    | Δ    | Δ    | Δ    | Δ    | Δ    |
| 9     | Mistakes/changes in the design documents provided by the employer.                              | 9          | ☐    | ☐    | ☐    | ☐    | ☐    | ☐    | ☐    |
| 10    | Delay in responding to contractor’s queries.                                                     | 9          | ☐    | ☐    | ☐    | ☐    | ☐    | ☐    | ☐    |
| 11    | Delay in the preparation process due to lack of resources, experience, management, etc.         | 9          | ☐    | ☐    | ☐    | ☐    | ☐    | ☐    | ☐    |
| 12    | Delay due to unforeseen conditions in shop drawings submission.                                 | 9          | Δ    | O    | Δ    | Δ    | O    | O    | O    |
| 13    | Delay in receiving the needed information to start the review of the work shop drawings process. | 9          | O    | O    | O    | O    | O    | O    | O    |
| 14    | Mistakes/changes in the generated shop drawings.                                                | 9          | ☐    | ☐    | ☐    | ☐    | ☐    | ☐    | ☐    |
Table 17(D). Correlation matrix

| Index | Categories of Delays and Causes |
|-------|---------------------------------|
| 1     | Delay in receiving the design criteria that are needed to start the design process. |
| 2     | Mistakes/changes in the design criteria provided by the employer. |
| 3     | Delay in responding to contractor’s queries. |
| 4     | Delay in approval stage. |
| 5     | Delay in the design process due to lack of resources, experience, management, etc. |
| 6     | Delay due to mistakes in the generated design documents. |
| 7     | Delay due to unforeseen conditions in design development. |
| 8     | Delay in receiving design documents that are needed to start the preparation of the workshop drawings process. |
| 9     | Mistakes/changes in the design documents provided by the employer. |
| 10    | Delay in responding to contractor’s queries. |
| 11    | Delay in the preparation process due to lack of resources, experience, management, etc. |
| 12    | Delay due to unforeseen conditions in shop drawings submission. |
| 13    | Delay in receiving the needed information to start the review of the work shop drawings process |
| 14    | Mistakes/changes in the generated shop drawings |
| 15    | Delay in responding to employer’s queries. |
| 16    | Delay in the approval process due to lack of resources, experience, management, etc. |
| 17    | Delay due to unforeseen conditions in approval stage |
| 18    | Changes due to mistakes/contradiction and/or constructability problems in the generated design documents. |
| 19    | Changes in construction procedure due to unforeseen site condition(s) |
| 20    | Changes in construction procedure due to soil investigation problem(s). |
| 21    | Changes in specifications in order to save time and/or cost. |
| 22    | Changes in specifications due to unavailability of materials. |

| Importance | TR29 | TR30 | TR31 | TR32 | TR33 | TR34 | TR35 | TR36 |
|------------|------|------|------|------|------|------|------|------|
| 1          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 2          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 3          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 4          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 5          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 6          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 7          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 8          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 9          | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 10         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 11         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 12         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 13         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 14         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 15         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 16         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 17         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 18         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 19         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 20         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 21         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |
| 22         | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  | 0.9  |

TR SUM | 9324 |

Weight | 0.102 0.133 0.131 0.139 0.145 0.114 0.120 0.116 |
| Index | Technical Requirements Summary                                                                 | T.R weight | Likert scale rating | AWAS level |
|-------|-------------------------------------------------------------------------------------------------|------------|---------------------|------------|
| TR1   | Materials accurate description and implementation methods.                                     | 0.131687   | 5                   | 4.895062   | 1          |
| TR2   | Specifications clarity and non-generalization.                                                   | 0.117284   | 5                   | 0.58642    |            |
| TR3   | Exact description of certain characteristics execution                                           | 0.12963    | 5                   | 0.648148   |            |
| TR4   | Possibility of applying the practical specifications of the project                             | 0.13786    | 5                   | 0.6893     |            |
| TR5   | Clarify the measurement methods used and conform to what is stated in other parts of the project | 0.117284   | 5                   | 0.58642    |            |
| TR6   | Characterization of test methods for construction materials.                                    | 0.104938   | 4                   | 0.419753   |            |
| TR7   | Avoid repetition of Specific work descriptions in two different formats or specifications conflict with other contract documents such as drawings. | 0.123457   | 5                   | 0.617284   |            |
| TR8   | Avoid using unknown standard specifications that may lead to misunderstanding                  | 0.13786    | 5                   | 0.6893     |            |
| TR9   | Avoid contrast and difference in units of measurement considered in different parts of the study. | 0.449275   | 5                   | 2.246377   | 4.449275   | 1          |
| TR10  | Clarify what includes the implementation of the unit of measure of work.                        | 0.311594   | 4                   | 1.246377   | 1.29386    | 3.219298   | 2          |
| TR11  | Selection of the appropriate measurement unit.                                                   | 0.23913    | 4                   | 0.956522   |            |
| TR12  | Review and check the technical conditions after the last printing, especially for methods and units of measurement and punctuation marks, etc. | 0.258772   | 5                   | 1.29386    | 3.219298   | 2          |
| TR13  | Divide project work into sections, chapters, and paragraphs properly fit the work received in the project. | 0.149123   | 3                   | 0.447368   |            |
| TR14  | Use punctuation correctly.                                                                     | 0.149123   | 3                   | 0.447368   |            |
| TR15  | Avoid using long and weak sentences.                                                             | 0.149123   | 2                   | 0.298246   |            |
| TR16  | Use understandable and known expressions                                                        | 0.149123   | 2                   | 0.298246   |            |
| TR17  | Avoid using general words.                                                                     | 0.144737   | 3                   | 0.434211   |            |
### Table 19. An Evaluation for Engineering-Related Delays

| Index | Technical Requirements Summary | T.R weight | Likert scale rating | AWAS level |
|-------|--------------------------------|------------|---------------------|------------|
| TR18  | Describe construction methods details. | 0.214286 | 2 | 0.428571 | 2.537815 | 3 |
| TR19  | Consider the execution ability method contained in the technical terms. | 0.247899 | 2 | 0.495798 |
| TR20  | Description of procedures security and public safety. | 0.277311 | 3 | 0.831933 |
| TR21  | Statement of implementation method clearly or in a manner that does not conflict with the rest of the tender documents. | 0.260504 | 3 | 0.781513 |
| TR22  | Approve the prices received with the required specifications. | 0.256318 | 3 | 0.512635 | 3.718412 | 2 |
| TR23  | Adequate and detailed price data. | 0.256318 | 3 | 0.768953 |
| TR24  | Avoid omission of the analysis or estimate of the price of the material or work required to implement an item. | 0.241877 | 5 | 1.209386 |
| TR25  | Avoid contrast and difference between the measurement unit used in pricing in both the Bill of Quantities and the Price Table or the specifications. | 0.245487 | 5 | 1.227437 |
| TR26  | Estimate the correct quantities of project works and avoid the exceeding, more than the specified percentage. | 0.524194 | 5 | 2.620968 | 5 | 1 |
| TR27  | Calculation of quantities based on detailed and final plans. | 0.475806 | 5 | 2.379032 |
| TR28  | Avoid the difference between the clauses and terms of the contract and the general conditions | 1 | 4 | 4 | 4 | 1 |
| TR29  | Design conforms to the wishes of the project owner or model design criteria. | 0.102317 | 5 | 0.511583 |
| TR30  | The design conforms to code requirements. | 0.133205 | 5 | 0.666023 |
| TR31  | Construction calculations match (Avoiding calculation errors). | 0.131274 | 3 | 0.393822 |
| TR32  | Avoid differences and inconsistencies among the different drawings (coordination flaw between different drawings: architectural, structural, civil ... etc.). | 0.138996 | 4 | 0.555985 | 4.220077 | 1 |
| TR33  | Operability/constructability problems • Lack of clarity on the construction and implementation mechanism, especially in non-recurrent special construction works, and in reinforcement and maintenance projects. | 0.144788 | 4 | 0.579151 |
| TR34  | Scale or dimensional errors | 0.1139 | 4 | 0.455598 |
| TR35  | The need for adequate architectural and construction details to complete the work as required. | 0.119691 | 4 | 0.478764 |
| TR36  | Compliance with standards and formalizations of drawing and avoiding errors. | 0.11583 | 5 | 0.579151 |
Table 20. Attributes assessment summary

| Engineering design of the project and preparing tender documents | AWAS | Assessment |
|---------------------------------------------------------------|------|------------|
| Specifications                                                | 4.89 | Level 1 (excellent) |
| Measurement units approved in the specifications              | 4.45 | Level 2 (satisfactory) |
| Printing                                                      | 3.22 | Level 3 (deficient) |
| Description of implementation technology and safety requirements| 2.54 | Level 1 (excellent) |
| Price and estimation                                           | 3.72 | Level 2 (satisfactory) |
| Bill of Quantities                                             | 5    | Level 3 (deficient) |
| Contract                                                      | 4    | Level 3 (deficient) |
| Engineering drawings                                          | 4.22 | Level 1 (excellent) |

Table 21. Actions to improve tender documents

| Attribute                                      | Detailed technical                                                                                                                                 |
|------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------|
| Printing                                      | Review and check the technical conditions after the last printing, especially for methods and units of measurement and punctuation marks, etc.        |
| Special technical specifications and writing  | Divide project work into sections, chapters, and paragraphs properly fit the work received in the project.                                            |
| Technical written documents                   | Use punctuation correctly.                                                                                                                                 |
| Description of implementation technology and safety requirements | Avoid using long and weak sentences.                                                                                                                                 |
| Price and estimation                           | Use understandable and known expressions                                                                                                                                 |
| Price and estimation                           | Avoid using general words.                                                                                                                                 |
| Description of implementation technology and safety requirements | Describe construction methods details.                                                                                                                                 |
| Description of procedure security and public safety | Consider the execution ability method contained in the technical terms.                                                                                                                                         |
| Statement of implementation method clearly or in a manner that does not conflict with the rest of the tender documents | Description of procedures security and public safety.                                                                                                                                                        |
| Price and estimation                           | Approve the prices received with the required specifications.                                                                                                                                               |
| Price and estimation                           | Adequate and detailed price data.                                                                                                                                                                          |
| Price and estimation                           | Avoid omission of the analysis or estimate of the price of the material or work required to implement an item.                                                                                              |
| Price and estimation                           | Avoid contrast and difference between the measurement unit used in pricing in both the Bill of Quantities and the Price Table or the specifications. |


6. Conclusions

Accordingly, through our study of claims in this research and through the QFD matrix, we can categorize these most influential claims to:

✓ Technical Documents Claims

Several errors may be made during the preparation of project technical documents, subsequently causing several claims. These claims are divided by nature into the following:

A. Claims of special technical specifications and writing.
B. Price claims and estimates.
C. Claims for the Bill of Quantities.
D. Contract Claims.

These claims are due to errors in the writing and preparation of these specifications. However, for different specification errors, we will review these errors by classifying them into the following:

A-1 Specifications errors include
- Misrepresentation of materials and methods of implementation.
- Ambiguity and generalization in specifications.
- Lack of descriptive information.
- It is not possible to apply the specifications in practice in the circumstances of the project for various reasons.
- Failure to clarify the methods of measurement used and the inconsistency with what is stated in the rest of the other tender documents of the drawings, tables, and quantities.
- Do not describe the testing methods for construction materials to obtain the necessary resistors or specifications.
- Reference to the use of a particular brand without mentioning information related to the quality or technical characteristics of the material.
- Duplicate a description of a particular work with two different shapes or conflicting specifications with other contract documents such as schemas.
- Use unknown standard specifications leads to misunderstanding.

A-2 Errors of units measurement approved in the specifications include:
- Variation and contrast in units of measurement in different parts of the study.
- Do not indicate what the implementation of the unit of measurement involves.
- Do not choose the appropriate unit of measurement.

A-3 Typographical errors:
- It results from the non-revision of the technical conditions and their revision after the last printing, especially concerning measurement methods, units, punctuation marks, etc.

A-4 Drafting and writing errors: It includes many errors, the most important ones
- Do not divide the project works into sections, chapters, and professional paragraphs properly-suited to work contained in the project.
- Do not use punctuation correctly (dot, semicolon, and comma).
- Use longitudinal and slender sentences and the frequent use of pronouns, making it difficult to understand sentence and purpose. It is preferable to use short and useful sentences that can perform the desired purpose.
- Use modern terms and terms that are not known and understood by everyone.
- Use general terms: best species, the best races, etc. Instead, it is preferable to use the language of numbers based on the physical and mechanical properties of the materials.

A-5 Claims related to the description of implementation technology and safety requirements

A-6 claims related to the description of implementation technology and safety conditions:
B- Claims due to price estimation errors or cost include (and may fall under other claims)

- Incompatibility of prices received with the required specifications.
- The price of vocabulary is insufficient and not detailed.
- Omission to analyze or estimate the price of a material or work required to implement an item.
- There are contrast and difference between the measurement unit used in the pricing in both the Bill of Quantities and the price table or the specifications.
- The price unit is not included in the price table and its incompatibility with technical conditions or specifications.
- The Issue of loading the price (a price or lump sum).

In most of the files or documents of these projects, we found differences between the contents of the various documents of the contract, which gives many possibilities for interpretation and interpretation, which led to the creation of various financial claims for the parties to the contract.

6.1. Limitations and Future Research

Despite the contributions of this work, there are two limitations. The first is that this study is ambitious in scope and scale but still subjected to restrictions in terms of time and access. The second limitation is that it would be beneficial to track further in time the implementation and to analyze its impact on the overall service quality. Regarding the potential future research, the author highly recommends the usage of QFD within innovative construction projects to prepare construction project documents to limit or reduce delays in construction projects.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References

[1] Eldin, Neil, and Verda Hikle. "Pilot study of quality function deployment in construction projects." Journal of construction engineering and management 129, no. 3 (2003): 314-329. doi:10.1061/(ASCE)0733-9364(2003)129:3(314).

[2] C. N. Madu. House of quality in a minute: Quality function deployment. Chi Publishers, 2006.

[3] Alrabghi, Leenah O. "QFD in software engineering." PhD diss., Kent State University, (2013).

[4] Amjad, A. "Comparative Analysis through Quality Function Deployment." Pakistan Research Journal of Management Sciences 1, no. 1 (2015).

[5] Zhang, Xingchen. "Integrating Lean Construction, BIM and Quality: A New Paradigm for the Improvement of Chinese Construction Quality." PhD diss., University of Bath, (2019).

[6] Haron, N. Z., and F. L. M. Kairudin. “The application of quality function deployment (QFD) in the design phase of industrialized building system (IBS) apartment construction project.” European International Journal of Science and Technology 1, no. 3 (2012): 56-66.

[7] Yang, Yi Qing, Shou Qing Wang, Mohammad Dulaimi, and Sui Pheng Low. "A fuzzy quality function deployment system for buildable design decision-makings." Automation in construction 12, no. 4 (2003): 381-393. doi:10.1016/S0926-5805(03)00002-5.

[8] John, Romeo, Andrew Smith, Sarich Chotipanich, and Michael Pitt. "Awareness and Effectiveness of Quality Function Deployment (QFD) in Design and Build Projects in Nigeria." Journal of Facilities Management 12, no. 1 (January 28, 2014): 72–88. doi:10.1108/jfm-07-2013-0039.

[9] S. R. Rabathi, D. A. Yunanto, A. Y. Bustomy, and S. Hasibuan, “Aviation Security Training Design Using Quality Function Deployment,” International Journal of Innovative Science and Research Technology 4, no. 12, (2019): 383-388.

[10] Toprakli, Abdurrahman Yagmur. "Evaluation of Courthouse Buildings within the Scope of Quality Function Deployment: Gaziantep Annex Courthouse Case." Gazi University Journal of Science Part B: Art Humanities Design and Planning 7, no. 3, (2019): 443-457.

[11] Khoder, Ranim Yehya. "Use of Quality Function Deployment in choosing suitable design specifications for users of residential Buildings in Lattakia-Syria." PhD diss., Tishreen University, (2017).

[12] Dodd, Alan M. Quality Function Deployment: A Method for Improving Contract Specifications in the US Corps of Engineers. Pennsylvania State University Park, Dept. of Civil Engineering, (1997).
[13] Haq, Saif Ul. "Impact of Safety and Quality Considerations of Housing Clients on the Construction Firms’ Intention to Adopt Quality Function Deployment: A Case of Construction Sector." International Journal of Economics and Management Engineering 13, no. 4 (2019): 412-418.

[14] Bernal, L., Utz Dornberger, A. Suvelza, and T. Byrnes. "Quality function deployment (QFD) for services." International SEPT Program, Leipzig, Germany (2009).

[15] Hsu, T. H., L. Z. Lin, and S. Y. Chiu. "Customer loyalty program based on quality function deployment with fuzzy linguistic preference relation." Iranian Journal of Fuzzy Systems 17, no. 1 (2020): 105-120.

[16] Dehe, Benjamin, and David Bamford. “Quality Function Deployment and Operational Design Decisions – a Healthcare Infrastructure Development Case Study.” Production Planning & Control 28, no. 14 (July 13, 2017): 1177–1192. doi:10.1080/09537287.2017.1350767.

[17] Haryo C. D., Raden Risang, Lalu Mulyadi, and Tiong Iskandar. “Delay Factors in Building Construction Project of State Elementary School.” Civil Engineering Journal 6, no. 3 (March 1, 2020): 523–530. doi:10.28991/cej-2020-03091488.

[18] Gargione, Luiz Antônio. "Using quality function deployment (QFD) in the design phase of an apartment construction project." In Proceedings IGLC, vol. 7 (1999): 357.

[19] Frey, Dan. "Quality Function Deployment." (2012).
Experimental Investigation of Self-compacting High Performance Concrete Containing Calcined Kaolin Clay and Nano Lime

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Abstract

The aim of this research is to investigate the effect of pozzolanic materials and nano particles on improve the strength characteristic by the properties of a self-compacting high-performance concrete that includes calcined clay with nano lime. In this study, two blends systems are worked on, they are the binary and the ternary systems. For binary mixtures, test samples were prepared from 5% CC, 10% CC, 15% CC and 3% NL by partial replacement of the cement weight. While ternary mixtures, samples were prepared from 5% CC 3% NL, 10% CC 3% NL and 15% CC 3% NL by partial substitution of cement weight. The tests conducted on mixes are fresh tests like slump flow diameter, V-funnel, L-box, and segregation resistance. The compressive strength test was determined at 7, 28 and 56 days. While splitting tensile strength tests at 7 and 28 days from the SCHPC produced in the study. It was concluded that the replacement of CC and NL in SCHPC binary mixes reduced the fresh results enough for SCHPC production and gave a general improvement in the compressive strength and splitting tensile strength properties of the SCHPC mixture. SCHPC with 10% CC partial replacement of cement showed higher values of compressive and splitting tensile strength, compared to the reference mixture of SCHPC for all days, thus it was considered the best. Whereas, the strength of the concrete mixtures in the ternary cement mixtures was better than the strength of the mixing and control mortar systems for the same replacement levels in 7, 28 and 56 days.

Keywords: Strength; Calcined Clay; Nano Lime; Self-Compacting High Performance Concrete (SCHPC).

1. Introduction

High-Performance Self-Compacting Concrete (SCHPC) is a special type of concrete. It combines the requirements of two types of concrete, namely, self-compacting concrete and high-performance concrete. Self-Compacting Concrete (SCC) is concrete that fills the mold and spreads during heavy reinforcement under the influence of its weight and does not require vibration [1]. Requirements for self-compact concrete include good filling ability, high passing ability, and ability to resist segregation. But it does not include durability and high resistance. Whereas, on the contrary, High Performance Concrete (HPC) is one of its most important requirements, and does not need its ability to filling and passing well [2]. To produce high performance self-compacting concrete, we need cement additives, which are useful for improving resistance, durability and segregation resistance. Also, we need high range water reducers, which are useful for achieving workability and passing ability [3]. The high-performance self-compacting concrete (SCHPC) has a high powder amount and little water content, while it has a lot of fine aggregate and little coarse aggregate compared to normal concrete [4].

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Self-Compacting High Performance Concrete (SCHPC) differ from conventional concrete in fresh and hardened states which are driven mainly by outstanding material components and combination proportions. This uses many special ingredients such as wide-range water reducing (HRWR) admixture for sufficient flow ability and a high volume of powder materials and/or Viscosity-Modifying Admixture (VMA) to achieve high segregation resistance, in addition to the basic materials used for Normal Vibrated Concrete (NVC). The proportions of component materials in SCHPC often differ materially from those in NVC [5]. In recent times, most of the students went to the field of nanoparticles due to their main effects in the field of concrete on (size, surface, interface, and quantum) [6, 7]. When adding nanomaterials to concrete mixtures, concrete performance and properties will improve due to the addition of nanomaterials. Some studies investigating the effect of adding nanomaterials to concrete are reported.

Özcan and Kaymak (2018) note that when adding metakaolin in different proportions 10%, 15% and 20% as a partial replacement with weight of cement to the self-compacting concrete mixture, the compressive strength will improve at ages 7, 28, 90 and 180 days to comparison with reference mixture [8]. Kumari et al. (2016), attempted to understand the effect of nano CaCO$_3$, nano TiO$_2$, and (nano TiO$_2$ + nano CaCO$_3$) upon the properties of the fly ash concrete with contents 0.5, 1, 1.5, 2, 2.5 and 3% by the weight of cement. From durability point of view, 2% substitution of nano particles demonstrated a very good resistance to Cl ion penetration and a high pH value. The workability of fresh concrete showed a decline in the slump value as the percentage of nanoparticles increased, compressive strength decreased with nano CaCO$_3$, while it increased in combination of nano TiO$_2$ and nano CaCO$_3$ at some percentages [9]. Wang et al. (2016), investigated the influence of nano CaCO$_3$ and nano silica on the hardening of cement-silica fume-fly ash UHSC. Results of the tests manifested that the flow ability of UHSC was decreased by incorporating nano CaCO$_3$ and nano silica content. The compressive strength increased at first then decreased, while the porosity value was vice versa. In regard, the calcium hydroxide content decreases with the increase of nano silica and raises with the nano CaCO$_3$ level increase [10].

Wu et al. (2016) studied nano CaCO$_3$ added to Ultra-High Strength Concrete (UHSC) in different proportions 1.6, 3.2, 4.8 and 6.4% as a partial replacement with cement weight. They observed that the compressive strength and flexural strength of UHSC increased compare to reference mixture at early and later ages. Except 6.4% nano CaCO$_3$ mixture. Whereas the strength decrease compare to the reference mixture [11]. Dadsetan and Bai (2017), investigated the mechanical and microstructural properties of self-compacting concrete (SCC) mixtures containing three Supplementary Cementitious Materials (SCM), namely metakaolin, ground granulated blast-furnace slag and fly ash. For the mixtures, cement was replaced by SCM at different levels 10, 20% for Metakaolin and 10, 20 and 30 % both GGBS and Fly Ash [12].

The mechanical properties were evaluated against a control mixture (without SCM). Metakaolin gave the most enhancing effect as a replacement material to cement on mechanical and microstructural properties of SCC at all ages. Faiz et al. (2017), studied the properties and microstructures of high volume fly ash cement paste incorporating with nano CaCO$_3$ and nano silica particles, nanoindentation results indicated the pozzolanic reaction, where the addition of 2% nano silica and 1% nano CaCO$_3$ increased the volume fraction of C-S-H gel and reduced the porosity [13]. Nanostructure showed that the high strength and high durable concrete can produce lower repair and maintains the requirements for concrete structure. Barkat et al. (2019), studied the effect of adding local metakaolin to the mechanical and rheological performance of self-compacting limestone cement. In this study, two types of cement were used, which are Portland Cement (PC) and Portland limestone cement (PLC), the local calcined clays were added at a temperature of 850 °C for 3 hours in different proportions 5, 10, 15, 20, 25% of the cement weight. The durability properties and porosity and water absorption were studied. Through the results, which showed that the porosity of water for concrete mixtures containing metakaolin at 28 days of age, decreased slightly compared to the reference mixture. The absorption of capillary water for mixtures containing metakaolin is lower compared to the reference mixture at ages 7, 90 days except for the mixture containing 10% metakaolin exposed to a high water absorption. This study achieved its goals through sustainable production SCHPC incorporating calcined pozzolanic materials (CC) and nanoparticles (NL) that could contribute to reducing cement demand, declining CO$_2$ emission rate, and make durable and eco-friendly SCHPC [14].

2. Materials and Mix Proportions

2.1. Materials

Cement

Type I cement was used in this study locally (Karasta). As its physical and chemical properties were identical to the Iraqi standard specifications (IQS NO.5./1984) [15].

Fine Aggregate

An important factor in producing high-performance self-compacting concrete that is taken into consideration is the amount of fine aggregate, its gradation and the shape of its particles. In this study, natural sand and locally present in
Al-Akhdar area was used as the fineness modulus 2.31 and it falls within the third gradient area according to the Iraqi standard specifications IQS NO.45/1984 [16]. Physical properties and sulfate content of fine aggregate as shown in table 1. While the grading of fine aggregate used is shown in Figure 1.

| Physical Properties          | Test Results | Limits of the Iraqi Specification (IQS NO. 45/1984) |
|-----------------------------|-------------|-----------------------------------------------------|
| Specific gravity            | 2.65        | --                                                  |
| Sulfate content %           | 0.3         | ≤ 0.50 %                                            |
| Absorption %                | 1.5         | --                                                  |
| Fineness Modulus            | 2.31        | --                                                  |

Figure 1. Grading of fine aggregate

Coarse Aggregate

The rounded, washed and locally available gravel was used in Al-Nabaei area with a maximum size 9.5 mm and it was (Specific gravity, absorption and sulfate content) 2.58, 0.5% and 0.03%, respectively, and according to the Iraqi standard specifications (IQS NO.45/1984)[16]. Figure 2 show the grading of coarse aggregate.

Super Plasticizer

High-range water-reducing additives were used in all concrete mixtures Called (Hyperplast PC200). It satisfies (ASTMC494) [17]. Where it was characterized by a Light yellow Colour, Specific gravity (s.g) 1.05±0.02 and air entrainment typically less than 2%.
Water

In this study, drinking water was used for mixing and curing purposes.

Calcined Clay

The kaolin clays that were used in this research are clays available in the western regions of Iraq. These clays were calcined to a temperature of 800 °C for two hours, to converts it to a pozzolan material named calcined kaolin clay (CC). As shown in Figure 3. Table 2 shows chemical compositions of Calcined Clay (CC). Whereas, the physical properties of (Specific gravity, Fineness m²/kg and Median particle size (μm)) are 2.59, 1640 and 14.3 respectively.

| Chemical Composition | Calcined Clay (%) |
|----------------------|-------------------|
| (SiO2)               | 54.7              |
| (Al2O3)              | 37.4              |
| (Fe2O3)              | 1.72              |
| (CaO)                | 0.84              |
| (MgO)                | 0.42              |
| (Na2O)               | 0.37              |
| (K2O)                | 0.54              |
| (SO3)                | 0.13              |
| (P2O5)               | 0.29              |
| (TiO2)               | 0.68              |

Loss on ignition (LOI) 2.91

Figure 3. Calcined clay used in this study

Nano-Lime

The nano-Lime used in this study was prepared by the Sky Spring nano materials company. Commercially available dry nano-CaCO₃ powder with a particle size of about 15–40 nm, 97.5% Calcite (CaCO₃) content, white colour and a surface area of 40 m²/g as shown in Figure 4.

Figure 4. Nano-Lime used in this study

2.2. Mix Proportions

In this paper, the European standard (EFNARC, 2005) was used to design concrete mixtures for self-compacting concrete [18]. The content of the raw materials used for concrete mixtures was for total binder content 485 kg/m³ and constant water/binder (w/b) ratio of 0.36. As for the super plasticizer, it is added in different proportions between 1.6 and 2.0% of weight of cement. Eight mixtures of concrete models were prepared with dimensions of 10× 10×10 cm and 10 diameter × 20 length cm for conducting compressive and splitting tensile strength tests of high-performance
self-compacting concrete. The first mixture (Reference-OP) is a reference mixture, four binary mixtures in which the cement was partially replaced with (calcined clays CC, nano lime NL) in different proportions 5% CC, 10% CC, 15% CC and 3% NL from the weight of cement. And three ternary mixture in which the cement was partially replaced with (calcined clays CC and nano lime NL) in different proportions 5% CC 3% NL, 10% CC 3% NL and 15% CC 3% NL from the weight of cement. After the molding process, it was curing 7, 28 and 56 days old. Table 3 and Figure 5 shows mix proportion of concrete mixture and experimental Program of the Research.

![Figure 5. Experimental Program of the research](image)

| Mix notation | Cement kg/m³ | Calcined clay kg/m³ | NanoCaCO₃ kg/m³ | Fine aggregate kg/m³ | Coarse aggregate kg/m³ | W/b % | Superplasticizer % |
|--------------|--------------|---------------------|-----------------|----------------------|------------------------|-------|-------------------|
| Reference-OP | 485          | ---                 | ---             | 850                  | 862                    | 0.36  | 1.6               |
| 5CC          | 460.75       | 24.25               | ---             | 850                  | 862                    | 0.36  | 1.6               |
| 10CC         | 436.5        | 48.5                | ---             | 850                  | 862                    | 0.36  | 1.6               |
| 15CC         | 412.25       | 72.75               | ---             | 850                  | 862                    | 0.36  | 1.6               |
| 3NL          | 470.45       | ---                 | 14.55           | 850                  | 862                    | 0.36  | 1.6               |
| 5CC 3NL      | 446.2        | 24.25               | 14.55           | 850                  | 862                    | 0.36  | 2.0               |
| 10CC 3NL     | 421.95       | 48.5                | 14.55           | 850                  | 862                    | 0.36  | 2.0               |
| 15CC 3NL     | 397.7        | 72.75               | 14.55           | 850                  | 862                    | 0.36  | 2.0               |

2.3. Tests and Curing Regimes

**Test of Fresh Properties**

Fresh tests for self-compacting high performance concrete (SCHPC) are important tests required to evaluate all the following three workability features of filling ability (flow ability and viscosity), passing ability, and resistance of segregation properties according to the [EFNARC, 2005][18]. However, there is no single test to measure the three characteristics together. In this experimental research, SCHPC's fresh properties were assessed by slump flow diameter, V-funnel, L-box and resistance tests to segregation. The slump flow tool was Abram's cone with a height of 30 cm, a diameter of 10 cm at the peak, and a diameter of 20 cm at the bottom. The slump flow is the diameter average (Dₘₐₓ and Dₜₑₚₚ) to the nearest 10 mm.

The test utilized to evaluate the filling ability SCHPC is the V funnel examine. Whereas, the test procedure and the
apparatus used are described in European self-compacting concrete guidelines, 2005. The funnel is filled with concrete from SCC with no pressure applied. Any excess concrete is removed from the top of the funnel using a straight edge. After a waiting period of (10 ± 2) seconds, we open the gate, and at the same time, the stopwatch starts. After that, we look inside the funnel and upon seeing visible areas we stop the watch. Stopwatch reading is reported as streaming time with V-funnel.

L-Box examination can be used to determine SCHPC’s passing ability to flow without segregation or blockage in the presence of reinforcing obstructions. As defined in the European self-compacting concrete guidelines, 2005. The vertical portion of the L box is filled with fresh SCC. Let the concrete stay for 1 minute in the vertical section. Concrete will be displayed during this time whether or not it is stable (Segregation). Second, we lift the slide gate and let the concrete flow out into the horizontal portion. Therefore, the height of the L-box was the average of the concrete height at the end of the horizontal portion to the concrete height at the beginning of the same section.

Stability test for sieves can be used to assess resistance of segregation (stability), the equipment and the test protocol as defined in the European guidelines, 2005. Resistance of segregation is the ability of a fresh blend to retain the initial, reasonably consistent distribution of constituent products. Approximately 10 liters of concrete were poured into a bucket, covered to avoid loss of moisture and allowed to settle for around 15 minutes. Then a concrete sample of 4.8 ± 0.2 kg was poured on a sieve of 5 mm with a diameter of 350 mm and left for 2 minutes to enable some mortar to move through and rest on a sieve pan with a weight scale. The segregation index was measured as the ratio of mortar weight compared to the weight on the sieve of the initial sample.

**Compressive Strength**

According to (BS. 1881: Part 116: 1989) [19] conducted the compressive strength test, Cubes (10x10x10) cm with a loading rate of 18 MPa/minute were tested using a hydraulic compression machine (2000) KN. At each test, an average of three cubes was adopted and tested at 7, 28 and 56 days.

**Splitting Tensile Strength**

The splitting tensile strength was applied using (ASTM C496-2004) [20] specification cylinders of 10x20 cm. The test involves the measurement of diametric compressive force along the length of a cylindrical concrete specimen at a rate of 1.4 MPa/minute until the failure occurs. At each test the average of two cylinders were taken and tested at 7 and 28 days.

**Curing Regimes**

Water curing for self-compacting High Performance concrete and is very essential due to the low water cement ratios employed. All of the specimens are demoulded 24 hours after casting and covered with plastic sheeting to preserve its moisture. Specimens are then cured in water at approximately 20 °C until testing is carried out.

3. Results and Discussion

3.1. Fresh Examination Results

Figures 6 and 7 present slump flow diameter and V funnel flow times, respectively, to show the level of SCHPC viscosity. Figures revealed the size of the slump flow diameter (mm) decreased and the height of the V-funnel flow period in the binary mixes 5CC, 10CC, 15CC and 3NL, by about 3.22, 5.16, 7.1 and 1.93% and 8.57, 18.57, 40 and 4.28% respectively, when compared with the reference mix. For the control concrete the lowest V-funnel flow times of 7.0s were determined whilst the 15CC combination had the maximum flow period of 9.8 s. The concrete generally became more viscous with CC in the binary system. However, utilizing NL substitution beyond 3 percent, the low viscosity rate resulting in low V-funnel flows of SCHPC mixtures. This reason could be due to the shape and size of the calcined kaolin clay particles which are long, hexagonal plates that create obstacles in the fresh mix and increase friction between the particles and also fine particle size of CC and Nanomaterials (NL), which have much larger surface areas that absorb water. The same results were proved by the other researchers [8, 21-23].

Whereas, in the ternary blends mixes 5CC-3NL, 10CC-3NL and 15CC-3NL, it can be observed that the slump flow diameter value declined and height V-funnel flow time, by about 6.45, 7.74 and 10.97% and 17.14, 40 and 62.86% respectively when compared with the reference mixture. The same trend was shown in the state of binary mixes with CC, while in ternary blends of mixtures incorporating CC and NL, the effect of replacement level more appeared. This is due to a result of the synergetic effect of the two reasons explained above. Similar trends were conducted by previous studies [12, 24]. Figure 8 displayed that the binary mixtures (5CC, 10CC, 15CC and 3NL) have lower passing ability compared to the control mixture, by about 2.53, 5.9, 7.38 and 2.21% respectively. The L-Box height ratio value diminished as partial substitution of cement increased. Because the influence of the proportion and fineness of calcined kaolin clay (CC) and nano lime (nano CaCO3), which reduces the ability of passing in the same way to reduce the filling ability. similar results were reported by [22, 25, 26]. But it does not agree with the study [27].
Whereas, it can be observed that this effect increases with the ternary mixtures 5CC-3NL, 10CC-3NL and 15CC-3NL by about 4.22, 7.49 and 10.76% compared to the unblended mixture 100% OPC. This is due to the result of the synergetic effect of the reasons explained above. No blocking or segregation phenomenon were found in the mixes at the time of the test execution. These results are similar to the results study [25]. From the previous works on fresh test of blended SCHPC, it can be observed that the replacement of CKC or NL, from the cement, rises in water demand because of their fineness and reactivity with cement [24, 28]. Figure 9 illustrated the results of the sieve stability test for binary and ternary blends of SCHPCs mixes, it can be seen that the binary mixtures (5CC, 10CC, 15CC and 3NL) have more segregation resistance compare with reference mix, as the percentage of segregation decreases with an increasing percentage of partial replacement. The percentage of decrease was about 13.33, 16.66, 25 and 5.83% for binary mixes above, respectively. Whereas, ternary mixtures (5CC-3NL, 10CC-3NL, and 15CC-3NL) have a higher segregation resistance compared with reference mixtures and binary mixtures of similar proportions. That is, the segregation ratio decreases further in the ternary mixes about 8.33, 22.5 and 31.66%, respectively. This may be due to impact of surface area increase of the nanoparticles, which leads to increased viscosity, thus increasing the segregation resistance. Similar results were with previous findings [29, 12].

### 3.2. Compressive Strength

After performing laboratory tests of high-performance self-compacting concrete mixtures in the laboratories of Babylon University/Faculty of Engineering, Department of Civil Engineering, the results shown in Table 4.

| Compressive strength MPa | Reference-OP | 5% CC | 10% CC | 15% CC | 3% NL | 5% CC 3% NL | 10% CC 3% NL | 15% CC 3% NL |
|--------------------------|--------------|-------|--------|--------|-------|-------------|-------------|-------------|
| 7day                     | 42.7         | 44.2  | 46.9   | 46.2   | 52.7  | 55.7        | 57.4        | 56.8        |
| 28day                    | 54.1         | 57.4  | 61.2   | 59.5   | 64.3  | 70.3        | 74.7        | 72.8        |
| 56day                    | 58.1         | 62.7  | 67.1   | 64.7   | 68.2  | 74.1        | 77.3        | 76.1        |
We note from the results obtained and shown in Table 4 and Figure 10, that the compressive strength in the binary mixtures (5%CC, 10%CC, 15%CC and 3%NL), has a higher compressive strength when compared to the values of the reference mixture, where the increase ratio was 3.51, 9.83, 8.19 and 23.41%, 6.1, 13.12, 9.98 and 18.85% and 7.91, 15.49, 11.35 and 17.38% at ages 7, 28 and 56 days, respectively. The reasons are due to the rapid pozzolanic reactions of calcined clay with Ca(OH)₂ to formation C-S-H and C-A-H as well as the large surface area of calcined clays compared to the surface area of the cement which also leads to the increase in reactivity. Thus, CC plays an important role in improving the bonds between cement paste and aggregate particles due to the formation of a dense microstructure in the ITZ, and it also increases the paste matrix, which could be attributed to both the physical and chemical advantages offered by CC. These results are in agreement with the previous study [8, 11, 25, 30].

Adding 3% nano lime to ternary mixtures 5% CC 3% NL, 10% CC 3% NL and 15% CC 3% NL increases the compressive strength about 30.44, 34.4 and 33%, 29.94, 38, 34.56% and 27.53, 33 and 30.98 for ages 7, 28, 56 days, respectively. As shown in Figure 11. Because lime nanoparticles contribute mainly to accelerating the hydration of early cement because of its high surface area. In addition, the nano lime works to fill in the voids and thus reduces pores and increases strength, these results are in agreement with the previous study [27, 28, 31].

3.3. Splitting Tensile Strength

Table 5 and Figure 12 shows that binary mixtures 5% CC, 10% CC, 15% CC and 3% NL have a higher indirect tensile strength when compared with reference mixture values. Where the increase rate was 3.61, 11.74, 9.63 and 22.59% and 6.77, 11.38, 9.44 and 14.04% in ages 7 and 28 days, respectively. The reasons are due to pozzolanic reactions and micro filling ability of CC, which reacts with CH and forms more C-S-H. This process leads to reduce in the porosity of concrete, therefore by improving the microstructure of concrete in ITZ and paste cement matrix tensile strength is increased. Besides, the large surface area of calcined kaolin clay leads to an increase in a reaction as well. These results are in agreement with the previous work [22, 25].

Adding 3% nano-lime to ternary mixtures as a partial substitution of cement weight 5% CC 3% NL, 10% CC 3% NL and 15% CC 3% NL the splitting tensile strength values increase more, by 26.8, 31.62 and 29.81% and 27.84, 33.89 and 29.29 % when compared to the reference mixture, for ages (7 and 28) days, respectively. As shown in Figure 13. Because nanoparticles contribute mainly to accelerating hydration of early cement due to its high surface area. In addition, the lime nanoparticle fills in the voids and thus reduces pores and increases strength. These results are in agreement with the previous work [11, 28].

| Splitting tensile strength MPa | Reference-OP | 5%CC | 10%CC | 15%CC | 3%NL | 5%CC 3%NL | 10%CC 3%NL | 15%CC 3%NL |
|-----------------------------|--------------|------|-------|-------|------|------------|------------|------------|
| 7day                        | 3.32         | 3.44 | 3.71  | 3.64  | 4.07 | 4.21       | 4.37       | 4.31       |
| 28day                       | 4.13         | 4.41 | 4.6   | 4.52  | 4.71 | 5.28       | 5.53       | 5.34       |

Figure 10. compressive strength values of binary mixes at 7, 28 and 56 days

Figure 11. compressive strength values of ternary mixes at 7, 28 and 56 days
3.4. Relationship between Compressive Strength and Splitting Tensile Strength

The Figure 14 shows the relationship between compressive strength and splitting tensile strength of all self-compact high performance concrete mixtures. This relationship was found for compressive strength ranging from 42.7 to 74.7 MPa and for splitting tensile strength from 3.32 to 5.53 MPa. The determination coefficient ($R^2$) for the best fit curve was 0.975, indicating an excellent relationship. This relationship was obtained for both ages (7 and 28 days) from concrete and shown in Equation 1. These results are in agreement with the previous study [23].

$$F_{sp} = 0.1237 f_{cu}^{0.8797}$$

(3)

6. Conclusion

In the binary blended mixes containing calcined kaolin clay (CC), fresh properties as (Slump flow diameter, L-box height ratio and segregation resistance) decreases with partial replacement of calcined kaolin clay increases. While, V-funnel time increase compared to the reference mixture. Adding 3% dosage of nano CaCO$_3$ to mixture lead to increase the viscosity of SCHPC (i.e. slump flow diameter, L-box height ratio, and segregation resistance) decreases while the V-funnel increases compared to the reference mixture. Fresh properties of ternary blended mixture (have CC and nano- CaCO$_3$ (NL)) more effect than binary mixes. in the binary mixes when adding calcined kaolin clays at rates 5% CC, 10% CC and 15% CC and nano lime at rate 3% NL as a partial replacement of the weight of cement leads to increase in the compressive strength and splitting tensile strength of the high-performance self-compacting concrete mixes in all ages compared to the reference mixture. Through the results of the examination it was found that the optimum percentage for replacing the calcined clay is 10% of the cement weight, as the best results were given. the addition of nano lime 3% of the cement weight in the ternary mixtures improves the compressive strength and splitting tensile strength of the concrete mixes at all ages. the strength of the ternary mixtures increases when compared with the binary mixtures as well as the reference mixture. In general, Splitting Tensile Strength (STS) of SCHPC mixes
appeared relatively the same trend with compressive strength, but the increase was at lower rates compared to that obtained in the compressive strength of SCHPC mixes.

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8. Conflicts of Interest

The authors declare no conflict of interest.

9. References

[1] Khayat, K. H. “Workability, Testing, and Performance of Self-Consolidating Concrete.” ACI Materials Journal 96, no. 3 (1999). doi:10.14359/632.
[2] Russell, H. G. “ACI defines high-performance concrete,” Concr. Int., vol. 21, no. 2, pp. 56–57, 1999.
[3] Khayat, K. H., and Y. Roussel. “Testing and Performance of Fiber-Reinforced, Self-Consolidating Concrete.” Materials and Structures 33, no. 6 (July 2000): 391–397. doi:10.1007/bf02479648.
[4] Persson, Bertil. “A Comparison Between Mechanical Properties of Self-Compacting Concrete and the Corresponding Properties of Normal Concrete.” Cement and Concrete Research 31, no. 2 (February 2001): 193–198. doi:10.1016/s0008-8846(00)00497-x.
[5] Okamura, Hajime, and Masahiro Ouchi. “Self-Compacting Concrete.” Journal of Advanced Concrete Technology 1, no. 1 (2003): 5–15. doi:10.3151/jact.1.5.
[6] Fang, Y., Yang, C. and Cheng, M. “An introduction to nanotechnology and nanomaterial (I),” CHINA SURFACTANT Deterg. Cosmet., vol. 33, no. 1; ISSU 191, pp. 55–59, 2003. doi:10.3126/hj.v5i0.1287.
[7] J. Wang and L. Wang, “Advances in the applied research of nano-material in concrete,” Concrete, vol. 11, pp. 18–21, 2004.
[8] Özcan, Fatih, and Halil Kaymak. “Utilization of Metakaolin and Calcite: Working Reversely in Workability Aspect—As Mineral Admixture in Self-Compacting Concrete.” Advances in Civil Engineering 2018 (August 29, 2018): 1–12. doi:10.1155/2018/4072838.
[9] K. Kumari et al., “Nanoparticles for enhancing mechanical properties of fly ash concrete,” Mater. Today Proc., vol. 3, no. 6, pp. 2387–2393, 2016.
[10] Wang, Dehui, Caijun Shi, Zemei Wu, Linmei Wu, Shuncheng Xiang, and Xiaoying Pan. “Effects of Nanomaterials on Hardening of Cement–silica Fume–fly Ash-Based Ultra-High-Strength Concrete.” Advances in Cement Research 28, no. 9 (October 2016): 555–566. doi:10.1680/jacdr.15.00080.
[11] Wu, Zemei, Caijun Shi, K.H. Khayat, and Shu Wan. “Effects of Different Nanomaterials on Hardening and Performance of Ultra-High-Strength Concrete (UHSC).” Cement and Concrete Composites 70 (July 2016): 24–34. doi:10.1016/j.cemconcomp.2016.03.003.
[12] Dadsetan, Sina, and Jiping Bai. “Mechanical and Microstructural Properties of Self-Compacting Concrete Blended with Metakaolin, Ground Granulated Blast-Furnace Slag and Fly Ash.” Construction and Building Materials 146 (August 2017): 658–667. doi:10.1016/j.conbuildmat.2017.04.158.
[13] Shaikh, Faiz U. A., Steve W. M. Supit, and Salim Barbhuiya. “Microstructure and Nanoscaled Characterization of HVFA Cement Paste Containing Nano-SiO2 and Nano-CaCO3.” Journal of Materials in Civil Engineering 29, no. 8 (August 2017): 04017063. doi:10.1061/(asce)mt.1943-5533.0001898.
[14] Barkat, A., S. Kenai, B. Menadi, E. Kadri, and H. Soualhi. “Effects of Local Metakaolin Addition on Rheological and Mechanical Performance of Self-Compacting Limestone Cement Concrete.” Journal of Adhesion Science and Technology 33, no. 9 (February 23, 2019): 963–985. doi:10.1080/01694243.2019.1571737.
[15] Iraqi Specification, “No. 5/1984, Portland Cement,” Cent. Organ. Stand. Qual. Control (COSQC), Baghdad, Iraq, 1984.
[16] Iraqi Specification, No.45, “Aggregates from Natural Sources for Concrete and Building Construction,” Iraqi Cent. Organ. Stand., 1984.
[17] ASTM C494/C494M-17, ”Standard Specification for Chemical Admixtures for Concrete”, American Society for Testing and Materials, (2017).
[18] EFNARC, “European Guidelines for Self-Compacting Concrete, Specification and Production and Use, Association House, UK.” 2005.
[19] B. S. Institutions, “Method for determination of compressive strength of concrete cubes,” London BS, 1881.

[20] ASTM C496. Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens,” ASTM Int. West Conshohocken, PA. (2004).

[21] Wang, Xin, Kejin Wang, Jinxin Li, Nishant Garg, and Surendra P. Shah. “Properties of Self-Consolidating Concrete Containing High-Volume Supplementary Cementitious Materials and Nano-Limestone.” Journal of Sustainable Cement-Based Materials 3, no. 3–4 (October 2, 2014): 245–255. doi:10.1080/21650373.2014.954155.

[22] Kannan, V. “Strength and Durability Performance of Self Compacting Concrete Containing Self-Combusted Rice Husk Ash and Metakaolin.” Construction and Building Materials 160 (January 2018): 169–179. doi:10.1016/j.conbuildmat.2017.11.043.

[23] Gholhaki, Majid, Ali kheyyroddin, Mohammad Hajforoush, and Mostafa Kazemi. “An Investigation on the Fresh and Hardened Properties of Self-Compacting Concrete Incorporating Magnetic Water with Various Pozzolanic Materials.” Construction and Building Materials 158 (January 2018): 173–180. doi:10.1016/j.conbuildmat.2017.09.135.

[24] Li, Wengui, Zhengyu Huang, Fangliang Cao, Zhihui Sun, and Surendra P. Shah. “Effects of Nano-Silica and Nano-Limestone on Flowability and Mechanical Properties of Ultra-High-Performance Concrete Matrix.” Construction and Building Materials 95 (October 2015): 366–374. doi:10.1016/j.conbuildmat.2015.05.137.

[25] Lenka, S., and K.C. Panda. “Effect of Metakaolin on the Properties of Conventional and Self Compacting Concrete.” Advances in Concrete Construction 5, no. 1 (February 25, 2017): 31–48. doi:10.12989/acc.2017.5.1.31.

[26] Gill, Anhad Singh, and Rafat Siddique. “Strength and Micro-Structural Properties of Self-Compacting Concrete Containing Metakaolin and Rice Husk Ash.” Construction and Building Materials 157 (December 2017): 51–64. doi:10.1016/j.conbuildmat.2017.09.088.

[27] Ge, Zhi, Kejin Wang, Renjuan Sun, Dawei Huang, and Yizhang Hu. “Properties of Self-Consolidating Concrete Containing Nano-CaCO3.” Journal of Sustainable Cement-Based Materials 3, no. 3–4 (March 31, 2014): 191–200. doi:10.1080/21650373.2014.903213.

[28] Shahidan, Shahiron, Bassam A Tayeh, A A Jamaludin, N A A S Bahari, S S Mohd, N Zuki Ali, and F S Khalid. “Physical and Mechanical Properties of Self-Compacting Concrete Containing Superplasticizer and Metakaolin.” IOP Conference Series: Materials Science and Engineering 271 (November 2017): 012004. doi:10.1088/1757-899x/271/1/012004.

[29] Barkhordari, Mohammad Sadegh, and Mohsen Tehranizadeh. “The Effect of Soil around the Basement Walls on the Base Level of Braced Framed Tube System.” Civil Engineering Journal 4, no. 9 (September 30, 2018): 2060. doi:10.28991/cej-03091139.

[30] J.-T. Ding and Z. Li, “Effects of Metakaolin and Silica Fume on Properties of Concrete.” ACI Materials Journal 99, no. 4 (2002). doi:10.14359/12222.

[31] Khotbehsara, Mojdeh Mehrinejad, Bahareh Mehdizadeh Miyandehi, Farzad Naseri, Togay Ozbakkaloglu, Faezeh Jafari, and Ehsan Mohseni. “Effect of SnO 2 , ZrO 2 , and CaCO 3 Nanoparticles on Water Transport and Durability Properties of Self-Compacting Mortar Containing Fly Ash: Experimental Observations and ANFIS Predictions.” Construction and Building Materials 158 (January 2018): 823–834. doi:10.1016/j.conbuildmat.2017.10.067.
Assessment of Waste Generation Rate in Teaching Hospitals of Metropolitan City of Pakistan

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Abstract

Hospital waste management is of vital significance owing to its contagious and hazardous nature as it can produce detrimental effects for both humans and the environment. This work aimed to examine types of waste with respect to waste generation rate in multiple teaching hospitals of metropolitan Lahore. A structured questionnaire survey, site visits, interviews and meetings were conducted in seventeen teaching hospitals. The results have shown that total hospitals average waste, infectious, non-infectious and waste generation rate in Lahore teaching hospitals were 38978 kg/day, 10789 kg/day, 28189 kg/day and 3.7 kg/bed/day, respectively. It is concluded that maximum waste generated in Mayo hospital, Jinnah hospital, Services hospital and Lahore general hospital was 16%, 12%, 12% and 10%, respectively, as per maximum patient’s visits. Positive linear correlation was between number of beds (P=0.917), number of accidents and emergency patients (P=0.75), infectious waste (P=0.998) and (P=1) with total waste. A straight line of linear regression was between (0.9966) infectious waste and (0.9995) general waste with average waste. Although, waste collection practices in these teaching hospitals were observed satisfactory but required training of doctors, nurses and hospital paramedical staff regarding infectious and general waste segregation. It is suggested that hospital staff, waste management and waste collection workers and respective waste management companies should be well trained and aware regarding infectious and non-infectious waste segregation, handling and disposing off procedures.

Keywords: Teaching Hospitals; Infectious and Non-infectious Waste; Waste Generation; Incineration; Waste Disposal.

1. Introduction

From last some decades, hospital waste management and disposal have become big issue globally. It is difficult to follow scientific and safe methods without disturbing environment. To maintain a sustainable safe environment, it is need of hour to emphasis on problem of hazardous hospital waste generation, waste segregation, waste collection and safe waste disposal approaches [1]. In general, hospital waste is a mixture of general waste, testing lab waste, medicinal chemicals, polymer or metallic vessels and pathological waste [2, 3]. Medicinal waste, contiguous waste and residential waste are main categories of hospital waste. Medicinal waste is stated as refuse generated during patient check-up, treatment and vaccination. Contiguous waste is specified as the part of waste relating to patients with transferable disease. Many time medicinal wastes are presumed as contiguous waste, as both are gathered collectively. Heterogeneous hospital waste is considered to be contiguous waste [4]. These contiguous wastes are the reason of various illnesses like cholera, plague, tuberculosis, ADIS (HIV), hepatitis and diphtheria [5].

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Various types of dangerous and non-dangerous hospital wastes consist of pathological, sharps, medicinal, nontoxic, chemical, radioactive, food scrap, empty containers and packages [6]. Infective waste has numerous categories for example human tissues and body parts, animal carcasses, syringes, blades, saws, drugs, vomits, urine, chemicals and fluid from laboratories that is the big source of HIV, Hepatitis B and C viral infections. Needle and sharp items infected with human blood spread these viruses commonly. Many other diseases that could be spread by interaction with hospital waste that is urinary tract infections, respiratory tract infections, wound infections, bacteremia and skin infections [7].

According to World Health Organization (WHO), used infected syringes are the prime source of two-million HIV patients and twenty-one million hepatitis patients throughout the world. This hazardous and non-hazardous waste consists of microbial germs that have a potential to become threat for people, workers and environmental health [5].

Numerous studies have been conducted to highlight the medical waste issues regarding its segregation at source point, collection on scientific bases, handling, and for its proper safe disposal in Poland, Canada, China as infectious waste directly endangers human being health and environment [8–10].

A study conducted in Korea about hospital waste management practices has reported that incineration of infectious waste is by far the most suitable method for waste management. However, it is linked with the risk of air pollution. Toxic air pollutants are not only a threat for biological species of the area but also for humans too as per the nature of polyvinyl chloride (PVC) products. So, waste reduction at source point, recycling and installation of toxic air pollution controllers are advised by authorities in Korea [11,12].

In Pakistan, according to the Hospital Waste Management Rules 2005, an Executive District Officer Health (EDO-H) is the responsible of environment and hospital waste management practices specially supervision and execution. In hospital facility level designated Waste Management Officer is responsible for implementation. According to a survey that was conducted in different districts of the provinces, 38% healthcare facilities is manage its waste under the supervision of committees headed by medical superintendent (MS) and selected waste management officer. While in 62% small healthcare or hospitals head nurse manages the hospital waste with the help of sanitary staff. Survey showed that one third out of visited facilities implemented on plans according to standards regarding waste collection, transportation and dispose-off, while one third implemented minimal plan according to standards while remaining had no plan [13]. Various studies conducted in Pakistan showed that approximately 2 kg/bed/day waste is generated in which 0.1-0.5 is considered as harmful waste. About 4-2000 kg/day waste is produced by different health related institutes out of which 75%-95% is non-infectious generated by hospital surroundings, organizational and managerial activities, while 10-25% is infectious waste and requires careful disposal [7]. Farooq and fellows research in 2017 showed better waste management situation in tertiary care hospitals Lahore. Waste collection and separation was done on source point. Waste transported through open trolley on-site and waste was stored in designated area. The study was concluded that majority of hospitals were working by following protocols but failure was in waste transportation in some hospitals [14]. WHO published a report in 2019 showing HIV outbreak in Larkana, Sindh Province, Pakistan. The report showed an alarming rate of 751 people infected with HIV positive. The reasons traced for this outbreak were the use of infected syringes, un-tested blood transfusion, improper hospital waste management and respective faulty governance in the area [15].

Toheed and fellows previously studied hospital waste management system in five teaching hospitals of Lahore city in 2012-2013. Waste segregation, on-site and off-site waste storage and transportation were not according to WHO recommendations and not followed laws as well as Pakistan Government regulations [16]. Toheed and fellows study did not show the comprehensive current condition of hospital waste management system. Present study covers seventeen teaching hospitals of Lahore city and results interpreted through statistical analysis. Magnin GPS Explorist 600 unit is used to prepare location map and to show infectious and non-infectious waste data.

In order to understand hospital waste management system and improve a management approach, it is necessary to study and analyze current practices in teaching hospitals. Therefore, in this study, types of waste were examined with respect to waste generation rate in multiple teaching hospitals in metropolitan, Lahore. A cross sectional study was conducted. Structured questionnaire data was interpreted through statistical analysis and pined out waste values on map.

2. Materials and Methods
2.1. Study Area

Lahore is the 2nd largest city of Pakistan and 1st biggest city of Punjab province. Geographically Lahore is situated in 31° 13’ and 31° 43’ N latitude and 74° 0’ and 74° 39.5’ E longitude [17]. According to 2017 census, population of Lahore is 11,126,285 [18]. There are many government, semi government, private and trust hospitals in Lahore. In order to know the hospital waste type and waste generation, seventeen Teaching hospitals were studied. All teaching hospitals were attached by Ministry of Health and Medical Education Department. Total infectious and non-infectious
waste generated in these teaching hospitals, was stated as hospital waste. Location map of the study area is shown in Figure 1.

![Location map of study area](image)

**Figure 1. Location map of study area**

### 2.2. Methodology

Infectious and non-infectious waste data were collected through questionnaire by visiting teaching hospitals of Lahore. Purpose of teaching hospitals visits was to get field observation and interviewed primarily. Collected data was cross checked through field observation. One-to-one interviews were conducted. To fill the designed questionnaire, information of waste types and quantity was collected from hospital waste collector staff, management staff, superintendents, doctors and nurses.

![Research Framework](image)

**Figure 2. Research Framework**
Quantity of hospital waste to be generated depends upon several factors such as size of hospital, healthcare type, occupancy rate of hospital beds, facilities, location and services provided by hospitals [19]. The designed questionnaire was clearly written. The data gathered from teaching hospitals include the following information:

- Number of beds in teaching hospitals;
- Outdoor patients visited per day;
- Indoor patients visited per day;
- Infectious waste (kg/day) of accidental and emergency patients;
- Outdoor patient’s infectious waste (kg/day);
- Admitted/hospitalized patients infectious waste (kg/bed/day);
- General waste (kg/day).

The designed questionnaire was given to interviewees before interview for clarity in answers. The infectious waste observations were taken by visiting the selected teaching hospitals waste generating, waste collection and authorized waste stored points. Observations were also taken through checking waste points and through monitoring. Outdoor doctors consulting rooms, operation theaters, laboratories, pharmacies, waste collection points and waste storage rooms were the waste checking point in selected hospitals. The study time duration was 3 days per hospital. Teaching hospital waste was monitored every day from generation source to storage point and till to incinerator plant where infectious waste finally incinerate. The purpose of the waste monitoring and observation was to ensure that the infectious waste was collected, handled, transported and disposed-off by following to SOPs.

Number of patients visited and admitted, and waste generated was monitored and recorded on record sheet per day. Collected data was tally with the hospital noted record too for the consistency and to avoid the ambiguity. Each hospital location coordinates were pinned out by using a Magnin GPS Explorist 600 unit.

2.3. Statistical Analysis

Data collected and filled questionnaires were used for calculating total infectious waste (kg/day), total general waste (kg/day), total waste generation rate (kg/bed/day), waste generation per hospital (%), and infectious waste per patient (%) through Microsoft Excel 2010. SPSS 16.0 were used for statistical (Pearson correlation coefficient, regression and t-test) data analysis.

3. Results

3.1. Total Waste Generation Rate

Seventeen teaching hospitals were surveyed and their infectious and non-infectious waste types and generation rates were measured. The teaching hospitals beds range from 30-2081, while total 11910447 numbers of out-patients and in-patients were visited per day. Teaching hospitals are normally general-purpose hospitals which facilitate all kind of patients, while few special kinds of diseases are handled in special-purpose hospitals like Punjab Institute of Cardiology, Punjab Institute of Mental Health, Punjab Dental Health and The Children Hospital & the Inst. of Child Health.

Total waste generation rate in these teaching hospitals were little different from each other depend on patients handling capacity and facilities provide in these hospitals. Total waste generated in these teaching hospitals was 38978 kg/day as shown in Table 1. Total waste generated rate was 3.7 Kg/bed/day.
Table 1. Waste generation rate in selected teaching hospitals of Lahore

| Sr. No. | Name of Teaching Hospitals                      | Beds | OPD visits p.a\(^1\) | A&E patients p.a | @kg/b/d 2.07 | Infectious 25% | OPD kg/p 0.075 | Infectious 80% | A&E kg/p 2.07 | Infectious 25% | General Waste 75% | Total Waste kg/day | Kg/bed/day |
|---------|-----------------------------------------------|------|----------------------|-----------------|-------------|----------------|----------------|----------------|----------------|----------------|-------------------|-------------------|-------------|
| 1       | Jinnah Hospital, Lahore                        | 1500 | 787,410              | 269,145         | 3,105       | 776            | 162            | 129            | 1,526          | 382              | 3506              | 4793             | 3.1953333   |
| 2       | Mian Munshi DHQ I Hospital                     | 149  | 473,474              | 287,991         | 308         | 77             | 97             | 78             | 1,633          | 408              | 1476              | 2039             | 13.684564  |
| 3       | Lady Atcheson Hospital                         | 200  | 142,000              | 29,037          | 414         | 104            | 29             | 23             | 165            | 41               | 440               | 608              | 3.04        |
| 4       | Mayo Hospital                                  | 2081 | 1,120,000            | 333,090         | 4,308       | 1,077          | 230            | 184            | 1,889          | 472              | 4694              | 6427             | 3.088419    |
| 5       | Services Hospital                              | 1196 | 1,520,638            | 329,274         | 2,476       | 619            | 312            | 250            | 1,867          | 467              | 3320              | 4656             | 3.8929766  |
| 6       | Sir Gunga Ram Hospital                         | 862  | 611,770              | 188,880         | 1,784       | 446            | 126            | 101            | 1,071          | 268              | 2167              | 2982             | 3.4593968  |
| 7       | Punjab Institute of Cardiology                 | 347  | 251,224              | 70,709          | 718         | 180            | 52             | 41             | 401            | 100              | 850               | 1171             | 3.3746398  |
| 8       | The Children Hospital & The Inst. Of Child Health | 684  | 550,000              | 0               | 1,416       | 354            | 113            | 90             | 0              | 0               | 1085              | 1529             | 2.2353801  |
| 9       | Lahore General Hospital                        | 1000 | 891,288              | 311,023         | 2,070       | 518            | 183            | 147            | 1,764          | 441              | 2912              | 4018             | 4.018       |
| 10      | Govt. Shandra Hospital                         | 300  | 418,167              | 255,157         | 412         | 155            | 86             | 69             | 1,447          | 362              | 1568              | 2154             | 7.18        |
| 11      | Govt. Muhammad Nawaz Shareef Hospital          | 200  | 845,075              | 128,682         | 414         | 104            | 174            | 139            | 730            | 182              | 893               | 1318             | 6.59        |
| 12      | Lady Willingdon Hospital                       | 235  | 79,908               | 15,497          | 486         | 122            | 16             | 13             | 88             | 22               | 434               | 591              | 2.5148936  |
| 13      | Said Mitha Hospital                            | 100  | 399,558              | 91,505          | 207         | 52             | 82             | 66             | 519            | 130              | 561               | 809              | 8.09        |
| 14      | Govt. Kot Khawaja Saeed Teaching Hospital      | 150  | 473,474              | 287,991         | 311         | 78             | 97             | 78             | 1,633          | 408              | 1477              | 2041             | 13.606667 |
| 15      | Govt. Mozang Teaching Hospital                 | 100  | 399,558              | 91,505          | 207         | 52             | 82             | 66             | 519            | 130              | 561               | 809              | 8.09        |
| 16      | Punjab Institute of Mental Health              | 1400 | 129,000              | 3,417           | 2,898       | 725            | 27             | 21             | 19             | 5                | 2193              | 2944             | 2.1028571  |
| 17      | Punjab Dental Health                           | 30   | 125,000              | 0               | 62          | 16             | 26             | 21             | 0              | 0                | 52                | 89               | 2.9666667  |
|         | Total                                          | 10,534 | 9,217,544            | 2,692,903       | 21,805      | 5455           | 1894           | 1516           | 15,271         | 3818             | 28189             | 38978            | 91.129793  |

3.2. Infectious and Non-infectious Waste Generation Rate

Total infectious and non-infectious wastes generated in teaching hospitals of research area were 10789 kg/day and 28189 kg/day, respectively. Percentage of total infectious and non-infectious wastes generated in teaching hospitals of research area has shown in Figure 3.
Figure 3. Percentage of waste generation in selected teaching hospitals of Lahore

Jinnah hospital generated total infectious and non-infectious waste 4793 kg/day (12%) with 3.2 kg/bed/day rate. Mian Munshi DHQ 1 hospital generated total waste 2039 kg/day (5%) with 13.7 kg/bed/day rate. Lady Aticheson hospital generated total waste 208 kg/day (2%) with 3 kg/bed/day rate. Mayo hospital generated total waste 6427 kg/day (16%) with 3.09 kg/bed/day rate. Services hospital generated total waste 4656 kg/day (12%) with 3.9 kg/bed/day rate. Sir Gunga Ram hospital generated total waste 2982 kg/day (8%) with 3.5 kg/bed/day rate. Punjab Institute of Cardiology generated total waste 1171 kg/day (3%) with 3.4 kg/bed/day rate. The Children hospital generated total waste 1529 kg/day (4%) with 2.2 kg/bed/day rate. Lahore General Hospital generated total waste 4018 kg/day (10%) with 4 kg/bed/day rate. Government Shahdra hospital generated total waste 1171 kg/day (3%) with 3.4 kg/bed/day rate. Government Shahdra hospital generated total waste 1218 kg/day (3%) with 7.1 kg/bed/day rate. Govt. Muhammad Nawaz Shareef hospital generated total waste 2041 kg/day (5%) with 13.6 kg/bed/day rate. Government Shahdra hospital generated total waste 809 kg/day (2%) with 2.09 kg/bed/day rate. Govt. Kot Khawaja Saeed teaching hospital generated total waste 2041 kg/day (5%) with 13.6 kg/bed/day rate. Govt. Kot Khawaja Saeed teaching hospital generated total waste 809 kg/day (2%) with 2.09 kg/bed/day rate. Punjab Institute of Mental Health generated total waste 2944 kg/day (8%) with 2.1 kg/bed/day rate. Punjab Dental Health generated total waste 89 kg/day (0.2%) with 2.9 kg/bed/day rate. Figures 4a and 4b show the concentration of infectious and non-infectious wastes of studied hospitals.

Figure 4(a). Concentration of infectious waste in study area hospitals
3.3. Percentage of Infectious Waste and Total Patient Visited Hospital

Figure 5 shows relationship between visited patients and infectious waste generating percentage in teaching hospitals. Approximately 15% patients visited Services hospital while 12% infectious waste generated per day. 12% patients visited Mayo while infectious waste generated 16% which is the biggest value of generation of infectious waste. Punjab Dental Health visited 1% patients and infectious waste generated 0.3% of total waste.

3.4. Percentage of Infectious Waste in the Total Hospital Waste Management System

Results have shown that in the studied hospitals, infectious waste was generated by 27% in Jinnah hospital, 28% in Mian Munshi, 38% in Lady Aticheson, 27% in Mayo hospital, 29% in Services, 27% in Sir Gunga Ram, 27% in Punjab Institute of Cardiology, 29% in The Children Hospital, 28% in Lahore General Hospital, 27% in Govt. Shahdra, 32% in Nawaz Shareef, 27% in Lady Willingdon, 30% in Said Mitha, 28% in Kot Khawaja Saeed, 31% in Mozang hospital, 26% in Mental Health, and 42% in Dental hospital as given Figure 6.
3.5. Statistical Analysis of Hospital Waste

Statistical Pearson correlation coefficient in Table 2 indicates that only number of outdoor patients shows (P=0.044) no linear correlation with total waste. There is perfect positive linear correlation between number of beds (P=0.917), number of accidents and emergency patients (P=0.75), infectious waste (P=0.998) and (P=1) with total waste. While in linear regression, there is a straight line between (0.9966) infectious waste and (0.9995) general waste with average waste. There is no pattern in (0.8409) number of beds, (0.5944) number of outdoor patient and (0.5619) number of accidental and emergency patients with average waste.

| Pearson Correlation Coefficient (r) | Simple Linear Regression (R^2) | t    | Sig. |
|-------------------------------------|--------------------------------|------|------|
| Number of beds                      | 0.917                          | 0.8409 | 4.149 | 0.001 |
| Number of Outdoor Patients          | 0.044                          | 0.5944 | 5.544 | 0     |
| Number of A&E Patient               | 0.75                           | 0.5619 | 5.035 | 0     |
| Infectious Waste                    | 0.998                          | 0.9966 | 5.452 | 0     |
| General Waste                       | 1                              | 0.9995 | 5.251 | 0     |

3.6. Infectious Waste Treatment

Two incinerators were present for teaching and public hospitals to incinerate the infectious waste with 3000kg/day capacity. One was working privately having 4000 kg/day capacity. The cost of incineration is PKR 80/kg.

4. Discussion

Metropolitan city Lahore Teaching Hospitals are working under Government sector. All Lahore teaching hospitals belong to ministry of Health. Its waste management governs (Lahore Waste Management Company (LWMC) that is a Semi Government department works under supervision of provincial Environmental protection department.

This study describes the waste types and generation rate. This statistics may be used as reference for controlling bodies and hospitals waste manager for better planning and improvement in any hospital infectious waste. This study may help the waste planners for waste treatment and its safe disposal.
Teaching hospital waste generation rate was totally varied from each other. The recorded values were very high than the studies have done in various other countries. Outside Pakistan, several studies have focused on the hospital waste management such as the India and Jordan where waste generation rate ranged between 0.5-2.0 and 0.52.2 kg/bed/day, respectively. Saudi Arabia and Kuwait have waste generation rates ranged from 0.03 to 3.78 and from 3.65 to 5.4 kg/bed/day, respectively. Moreover, in European hospitals, the generation rates of medical waste were 3.9 kg/bed/day in Norway, 4.4 kg/bed/day in Spain, 3.3 kg/bed/day in United Kingdoms and 2.5 kg/bed/day in France [20]. Similarly, the average waste generation rate of hospitals (2.79-3.86 kg/bed/day) in Taiwan was much high as compared to Poland (2.6 kg/bed/day), Japan (0.25 kg/bed/day) and Korea (0.48 kg/bed/day) [11, 21, 22]. This may be due to various factors as numerous studies have been conducted to find the factors that affect average waste production per day in hospitals and clinics [23–25].

During the research, it was observed that infectious waste generation rate was high with increasing the number of patients visited teaching hospitals. Present study demonstrates that waste generation in teaching hospitals of Mayo hospital, Jinnah hospital, Services hospitals and Lahore General hospital were comparatively high than other studied teaching hospitals. The variability of hospital waste generation can be indicative the fact, that total beds, or other factors may affect for example number of outpatient, inpatient, and patients came due to accidents or in emergency. Lab facilities, surgeries performed per day, number of departments, wards and hospital type may also be the reasons of maximum waste generation in these hospitals. Ceraro and Belgioirno (2015) has documented that hospital waste generation rate in different countries depends on several factors like provision of hospital services, disposable or hospital reusable instruments quantity, and laws, regulations and policies enforced there [26].

In the present study, waste is divided in to two parts; infectious and general waste according to Hospital waste Management Rules, 2005 Pakistan. Approximately in all teaching hospitals, the proportion of infectious to general waste was very high than WHO recommendation. According to WHO, the waste generated by hospitals and healthcare are general waste, estimated 75-90% (85%) while remaining 10-25% is hazardous and 10% waste is hazardous infectious. High rate of infectious waste might be due to different factors for instance improper infectious waste segregation, less training programs and lack of planning. Infectious waste treatment and disposal cost significantly can be reduced by implementing the educational programs and proper planning of waste minimization [27]. To enhance segregation efficiency and reduce inappropriate use of containers, the containers should be placed at proper place and labeled carefully [28]. A study was conducted in Greece in which nurses were approached specially for waste segregation, waste minimization price and pollution avoidance planning and execution process. Results showed that nurses were already well aware about waste segregation but facing hurdle in waste minimization during their duty [29]. Niamul Bari and fellows conducted research in 2019, to evaluate information about the practices related to waste segregation. In many hospitals collection of wastes in Rajshahi City from source of generation is not properly performed and not used colour containers as well [30].

Moreover, regression analysis, pearson correlation coefficient and t-test statistics were used for better understanding. There is perfect positive liner correlation between number of beds, number of accidents and emergency patients, and infectious waste with total waste. Only number of outdoor patients has no linear correlation with total waste. Sanida et al. (2010) used these statistical analyses to evaluate relationship of number of beds, out-patient, accidents and emergency patients, infectious and general waste with total waste. In addition, Eslam et al. (2017) used Pearson correlation coefficient and concluded that there was significance difference among private and governmental hospitals, with reference of infectious and sharp waste generation rates (P=0.027). Significant differences between specialized and general hospitals were also showed in group of non-hazardous waste (P=0.039), infectious waste (P=0.001) and total waste generation rate (P=0.02). To obtain accuracy in researches results, various hospital infectious and noninfectious studies were concluded with statistical analysis [31, 32].

Similarly, regression analysis results have shown that there are straight fit lines of general and infectious wastes with total waste as shown in Figure 7. It has direct positive relationship. Number of beds, number of outpatient and number of accidents and emergency patient’s results are scattered as shown in Figure 7. These results show no patterns and have weak linear relationship.
Issam A Al-Khitab conducted a research for secure outcome of waste management plan. For the estimating the general waste, total hazardous waste and daily total waste of hospital, multiple-variable regression model set was used. The purpose of the regression model used was that it may help in planning of waste management and to satisfy the
increasing waste treatment and disposal demand of hospitals [33]. Similarly a study was conducted at Upazila level Bangladesh in 2014 to identify the generation of hospital waste from the medical services. Where the amount of hospital waste was positive correlated with the number of occupied beds ($R^2 = 0.898$) and with the number of patients ($R^2 = 0.785$) [34]. Moreover, a study was done by Mohee about healthcare institution Mauritius in 2004, which revealed the exercise of regression relationship between number of beds and amount of hazardous waste. The $R^2$ value was 0.9551. The results $y=0.0006$ where $y$ was the hazardous waste amount per bed per day. While $x=0.19$ was obtained, $x$ was used for number of occupied bed. Author concluded that this equation is applicable for large hospitals [35].

The waste generator entities are responsible for ensuring proper waste disposal. In the present research the teaching hospitals have proper waste management plans and team. Practices regarding infectious waste disposal or incineration were quite good according to the waste management plan and were similar in all the hospitals. The wastes of 165 hospitals are brought at this incinerator for disposal from in and outside of Lahore [36]. An investigation was done by Abdullah and fellows in north Jordan in 2007. The purpose of the investigation was to know the level of segregation, treatment and disposal option in selected hospitals. The results reported similarly like current study regarding segregation in which infectious hospital waste was done improperly. While for infectious waste treatment condition was so poor. Incinerators were used only for 48% infectious waste. None incinerator working was met with Ministry of health regulations. Their 57% liquid waste was thrown in municipal drain while left liquid waste was collected in septic tanks [37]. Yong and fellows conducted a research in China, Nanjing in 2009 and, concluded that only incineration technology was being used by waste disposal companies [38]. Dioxin and furans are released from hospital waste incinerators due to polyvinyl chloride (PVC) plastic and affect human reproduction, development and immune system function, also cause cancer and environmental impacts. In spite with strict environmental laws and regulations lot of criticisms have been received. Responsible authorities should pay attention on this issue because of criticism by public and researchers [12, 38, 39].

According to the Hospital Waste Management Rules (2005), under the provision of Pakistan Environmental Protection Act (1997), each hospital is the responsible for its proper waste management (segregation, handling, storage and transportation) till its final disposal [40]. These guidelines are applicable to all hospitals, clinics, dispensaries, maternity centers, dental clinics, pathological laboratories, blood banks, nursing homes, research institutes and other health care facilities [6]. Similarly a research was conducted by Goren in 2011 in Turkey, where to avoid the danger of medicinal waste to workers and people, Medical Waste Control Regulation were used to express waste management system. To minimize the environmental pollution sterilization plant extension was under consideration [41]. Proper awareness and knowledge has become essential requirement for proper hospital waste management [42].

5. Conclusion

The aim of this study was to examine types of waste with respect to waste generation rate in multiple teaching hospitals in metropolitan Lahore. The research was conducted to quantify the waste generated from different hospitals. The total hospital’s average waste, infectious, non-infectious and waste generation rate in surveyed teaching hospitals were 38978 kg/day, 10789 kg/day, 28189 kg/day and 3.7 kg/bed/day, respectively. Amount of general and infectious waste are high with the number of patients visit to the hospital. It can be due to the various factors including the facilities and services that are provided by the hospitals. Percentage of infectious waste is far high in 42% in Dental hospital, 38% in Lady Aticheson, 32% in Nawaz Shareef, 31% in Mozang hospital, and 30% in Said Mitha than WHO guidelines. Others remaining hospitals infectious waste values were between 26-30% that values are also so high. High percentages can be the reason of improper segregation. All collected data have shown that nearly all hospitals do not segregate their infectious waste accurately. In all studied teaching and public hospitals, infectious wastes is being collected and transported to incinerator plant and being incinerated there regularly according to the laws and regulations.

It was observed that public teaching hospitals are working and providing better practices and services under supervision of government following the national laws and regulations for hospital waste management. It is suggested that periodic education and training programs should be conducted by government for workers related to infectious and non-infectious hospital wastes segregation and also pay attention on release of hazardous gases from incinerators.

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7. Conflicts of Interest

The authors declare no conflict of interest.
8. References

[1] Ghosh, Sadhan Kumar, ed. “Energy Recovery Processes from Wastes” (2020). doi:10.1007/978-981-32-9228-4.

[2] Zafar, Rabeea, Syed Shahid Ali, Zaheer Uddin, and Maqsood A. Khan. “A Case Study of Hospital Waste Management in Balochistan and Its Impact on Health and Environment.” Research Journal of Environmental and Earth Sciences 5, no. 2 (February 20, 2013): 98–103. doi:10.19007/rjees.5.5644.

[3] Khan, Khushbukhat, Sidra Shaheen, Huma Iqbal, Ghazala Mushfaq Raja RohinaArif, Madeeha Khalil, Amna Munawaa, Maryem Batoold et al. “Assessment of Waste Management Practices in Hospitals of Islamabad and Abbottabad-Pakistan.” Pure and Applied Biology 9, no. 1 (March 10, 2020). doi:10.19045/bspb.2020.90033.

[4] Altin, S., A. Altin, B. Elevli, and O. R. H. A. N. Cerit. “Determination of hospital waste composition and disposal methods: a case study.” Polish Journal of Environmental Studies 12, no. 2 (2003): 251-255.

[5] Hameed, Kashif, Omar Riaz, Muhammad Nasar u Minallah, and Huma Munawar. “Types of Hospital Waste and Waste Generation Rate in Different Hospitals of Faisalabad City, Pakistan.” Journal of Basic & Applied Sciences 13 (July 4, 2017): 386–391. doi:10.6000/1927-5129.2017.13.63.

[6] Rasheed, Shahida, Saira Iqbal, Lubna A. Baig, and Kehkashan Mufti. "Hospital Waste Management in the Teaching Hospitals of Karachi." JPMA 55 (2005): 192.

[7] Arshad, Nosheen, Shamail Nayyar, Fatima Amin, and Khawaja Tahir Mahmood. "Hospital waste disposal: a review article." Journal of Pharmaceutical Sciences and Research 3, no. 8 (2011): 1412.

[8] Wajs, Jan, Roksana Bochniak, and Aleksandra Golabek. “Proposal of a Mobile Medical Waste Incinerator with Application of Automatic Waste Feeder and Heat Recovery System as a Novelty in Poland.” Sustainability 11, no. 18 (September 12, 2019): 4980. doi:10.3390/su11184980.

[9] Yan, Chen, Zhang Kai, Xu Jiyun, Shao Zheru, and Ji Ru. “Analysis of Cooperative Disposal of Medical Waste Treatment and Municipal Solid Waste Incineration.” IOP Conference Series: Earth and Environmental Science 295 (July 25, 2019): 012052. doi:10.1088/1755-1315/295/2/012052.

[10] Aboelnour, Amal, and Manal H. Abuelela. “Increase Adherence to Waste Management Policy at Healthcare Facility in Egypt.” Bulletin of the National Research Centre 43, no. 1 (February 18, 2019). doi:10.1186/s42269-019-0065-2.

[11] Jang, Yong-Chul, Cargro Lee, Oh-Sub Yoon, and Hwidong Kim. “Medical Waste Management in Korea.” Journal of Environmental Management 80, no. 2 (July 2006): 107–115. doi:10.1016/j.jenvman.2005.08.018.

[12] Thornton, J., M. McCallly, P. Orris, and J. Weinberg. “Dioxin prevention and medical waste incinerators.” Occupational Health and Industrial Medicine 1, no. 36 (1997): 11.

[13] Punjab Health Sector Reforms Support Project. Environmental and medical waste management plan. 2013.

[14] Mt, Farooq. "Assessment of Hospital Waste Management Protocols in Tertiary Care Hospitals of Lahore." Biomedica 33, no. 2 (2017).

[15] Ilyas, F. and J. Ikram. “Special report: The making of an HIV catastrophe - Pakistan - DAWN.COM.” (2019): 1–10.

[16] Toheed, Rakshanda, TALHA BIN AYUB, and Saba Mumtaz. "Hospital Waste Management in Teaching Hospitals of Lahore System Assessment Using New Tool." Pakistan Journal of Medical & Health Sciences 10, no. 2 (2016): 377-379.

[17] Naqvi, Syed, Syed Kazmi, Saima Shaikh, and Maryum Akram. “Evaluation of Prevalence Patterns of Dengue Fever in Lahore District through Geo-Spatial Techniques.” Journal of Basic & Applied Sciences 11 (January 19, 2015): 20–30. doi:10.6000/1927-5129.2015.11.04.

[18] Baiwa A. 6th Population and Housing Census Provisional Results and Issues. (2018).

[19] Tabasi, Rahele, and Govindan Marthandan. "Clinical waste management: A review on important factors in clinical waste generation rate." International Journal of Science and Technology 3, no. 3 (2013): 194-200.

[20] Abd El-Salam, Magda Magdy. “Hospital Waste Management in El-Beheira Governorate, Egypt.” Journal of Environmental Management 91, no. 3 (January 2010): 618–629. doi:10.1016/j.jenvman.2009.08.012.

[21] Zimmermann, Agnieszka, and Robert Szyc. "Medical Waste Management in Poland the Legal Issues." Polish Journal of Environmental Studies 21, no. 4 (2012).

[22] Cheng, Y. W., F. C. Sung, Y. Yang, Y. H. Lo, Y. T. Chung, and K-K. Li. "Medical waste production at hospitals and associated factors." Waste Management 29, no. 1 (2009): 440-444. doi:10.1016/j.wasman.2008.01.014.
[23] Alhumoud, Jasem M., and Hani M. Alhumoud. "An analysis of trends related to hospital solid wastes management in Kuwait." Management of Environmental Quality: An International Journal (2007). doi:10.1108/14777830710778274.

[24] Marinković, Natalija, Ksenija Vitale, Nataša Janve Holcer, Aleksandar Džakula, and Tomo Pavić. "Management of hazardous medical waste in Croatia." Waste management 28, no. 6 (2008): 1049-1056. doi:10.1016/j.wasman.2007.01.021.

[25] Sanida, G., A. Karagiannis, F. Mavridou, D. Vartzopoulos, N. Moussipoulos, and S. Chatzopoulos. "Assessing generated quantities of infectious medical wastes: A case study for a health region administration in Central Macedonia, Greece." Waste Management 30, no. 3 (2010): 532-538. doi:10.1016/j.wasman.2008.11.019.

[26] Cesaro, A., and V. Belgiorno. "Medical waste generation and management in different sized facilities." In Proceedings of the 14th International Conference on Environmental Science and Technology, Rhode, Greece, pp. 3-5. (2015).

[27] Eslami, Akbar, Parviz Nowrouz, and Samira Sheikholeslami. "Status and challenges of medical waste management in hospitals of Iran." Civil Engineering Journal 3, no. 9 (2017): 741-748. doi:10.21859/cej-030910.

[28] Theofanidis, D., A. Fountouki, F. Vosniakos, N. Papadakis, and K. Nikoalou. "Sustainable management of hospital waste: The view of Greek nurses." J. Environ. Prot. Ecol 9 (2008): 391-403.

[29] Minoglou, Minas, Spyridoula Gerassimidou, and Dimitrios Komilis. "Healthcare waste generation worldwide and its dependence on socio-economic and environmental factors." Sustainability 9, no. 2 (2017): 220. doi:10.3390/su9020220.

[30] Mohee, Romeela. "Medical wastes characterisation in healthcare institutions in Mauritius." Waste management 25, no. 6 (2005): 575-581. doi:10.1016/j.wasman.2004.10.003.

[31] Cheremisinoff PN, Shah MK. Hospital waste management. vol. 22. 1990.

[32] Goren, S. "Evaluation of waste management systems in Turkey." Journal of Environmental Protection and Ecology 12, no. 2 (2011): 621-628.

[33] Akter, Kazi Shamima, and Shaikh Mohammad Shamim Reza. "Awareness on Medical Waste Management and Occupational Health Safety among the Employees Related to Medical Services at Upazila Level in Bangladesh." Journal of Environmental Treatment Techniques 7, no. 3 (2019): 282-288.
Structural Behavior of Pipelines Buried in Expansive Soils under Rainfall Infiltration (Part I: Transverse Behavior)

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Abstract

Landslides, fault movements as well as shrink/swell soil displacements can exert important additional loadings on soil buried structures such as pipelines. These loadings may damage the buried structures whenever they reach the strength limits of the structure material. This paper presents a two-dimensional plane-strain finite element analysis of an 800 mm diameter water supply pipeline buried within the expansive clay of the Ain-Tine area (Mila, Algeria), considering the unsaturated behavior of the soil under a rainfall infiltration of 4 mm/day intensity and which lasts for different time durations (8, 15 and 30 days). The simulations were carried out using the commercial software module SIGMA/W and considering different initial soil suction conditions P1, P2, P3 and P4. The soil surface heave and the radial induced forces on the pipeline ring (i.e., Axial $F_A$, Shear $F_S$ forces and bending moments $M_B$) results indicated that following the changes of suction the rainfall infiltration can cause considerable additional loads on the buried pipeline. Moreover, these loads are proportionally related to the initial soil suction conditions as well as to the rainfall infiltration time duration. The study highlighted that the unsaturated behavior of expansive soils because of their volume instability are very sensitive to climatic conditions and can exert adverse effects on pipelines buried within such soils. As a result, consistent pipeline design should seriously consider the study of the effect of the climatic conditions on the overall stability of the pipeline structure.

Keywords: Finite Element Analysis; Unsaturated Soil; Buried Pipeline; Rainfall Infiltration; Suction; Radial Internal Forces.

1. Introduction

Buried pipelines are important lifeline infrastructures used by many countries and companies to transport fluids (i.e., water or gas) to remediate the strong hydraulic and energy resources inequalities over the world. The geotechnical and structural engineers faced many challenges to safely and durably design and build these budget-consuming projects. Landslides, fault movements as well as shrink/swell soil displacements can exert important additional loadings on structures especially for pipelines buried within expansive soils. These loadings may damage these structures or disturb their normal operations whenever their magnitudes reach the strength limits of the structure material. 

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In recent decades, professional and academic forensic surveys have revealed that the unsaturated behavior of expansive soils is the main cause of the reported damages that occurred on many types of structures such as lightweight structures and buildings [1, 2], water transport canals used in agriculture activities [3]. Severe damages were reported also on buried pipelines [4, 5] in many parts of the world such as arid and semi-arid regions, as a result of deformations induced by volume changes that characterize expansive soils [6, 7].

It is known that arid and semi-arid areas, are usually characterized, by a deep water table and by extremely dry surface soils which bear negative pore water pressures (suction) [8] and which are extremely sensitive to wetting and drying. When these soils are expansive, the supply of a tiny quantity of water may develop within these soils important swelling deformations as well as important swelling pressures when the deformations are blocked. The mechanical characteristics (i.e., shear strength and deformation parameters) of the soils located above the water table are inversely related to the moisture content which increases the sensitivity of these soils to the climate conditions such as rainfall [9–11]. Han et al. [12] describe the suction as an energy potential that takes the form of a tension stress which is exerted on the soil skeleton, this potential keeps, together, the soil particles in packets, offering more resistance of the soil against deformations.

Rainfall precipitation or irrigation activities provide a downward water flow in the expansive soil increasing its moisture content and reducing its suction simultaneously, and consequently provokes volume changes which are measured as vertical displacements on the ground surface of such types of soil [10, 13, 14]. The volume changes that occur in the pipeline embedding soil provoke loading forces on the structure of the pipeline that can endanger the stability of the lifeline infrastructure. The unsaturated behavior effects on pipeline response have been analyzed both experimentally and numerically as well as under different loading conditions (i.e., landslides, faults movements ...).

Experimental large scale tests have been conducted by Randeniya et al. [15] to investigate deformations of a steel pipeline buried in an unsaturated clayey soil under different saturation conditions; they found that the backfill soil’s degree of saturation can considerably control pipeline deformations. Full-scale tests and finite element simulations were carried out by Robert et al. [16]. They found that the unsaturated soil strength and stiffness increase with soil suction and externally imposed ground movements increase lateral loads on pipelines. Huang et al. [17] have conducted a full scale analysis of the effect of the frost heave on a 105m long and 0.9m diameter chilled gas pipeline with 35m long was buried in permafrost in Alaska. They found that the pipelines buried in the arctic region suffer damages due the induced soil vertical movements which principally take the form of bending actions causing strains on the pipeline bodies.

In which concern numerical studies, a finite element analysis has been conducted by Rajeev and Kodikara [18] and Robert and Soga [19]. The obtained results showed that the reduction in suction due to the increase in moisture content provokes an increase of the soil loading on the buried structures such as pipelines. In this direction, an interesting study about the background of pipelines have been undertaken by Al-Khazaali et al. [20] that can be used as a good platform to do pipeline-soil interaction researches in unsaturated mediums. Moreover, the response of buried pipelines under strike-slip fault movements have been studied by Vazouras et al. [21] and Oghabi et al. [22]. Saadeldin et al. [23] investigated through a parametric study the influence of the moisture content variations on the longitudinal movements of a hypothetical pipeline of 6 m length and 0.15 m diameter that occurred due to change in suction by considering different boundary conditions. The transverse soil-pipeline system deformations associated with trenching was studied by Al-Khazaali et al. [20] considering the effect of the groundwater table (GWT) level and that of the depth of excavation. This study helps to define the safe combination (GWT, Depth) when trenching near to buried pipeline in unsaturated soils.

All the previous studies highlighted the importance of taking into consideration the unsaturated behaviour of the expansive embedding soils which can cause important additional loadings on the buried structures considering the longitudinal response of the pipelines.

To address the substantial hydrological inequalities that characterize the eastern side of Algeria, more than 600 km of water supply pipelines supplying water from the Beni-Haroun dam, were buried in the Mila basin (Mila province, Algeria), a region known by its semi-arid climate [24] and famous as a highly sensitive region to shrink/swell movements [25]. In recent years, many of the water transport pipelines have suffered repetitive damages in north of Algeria taking the form of leakage points along the pipeline route. During the rainy season the problem worsens and the leakage rate recorded increases. Unfortunately, after the construction of the Beni-Haroun dam in 2006 (i.e., the largest dam in Algeria, Mila province), the Algerian Ministry of Water Resources received several investigation reports on the damages that occurred on the water supply pipelines which are experiencing many leakage points along their routes. The site named Aine-Tine is one of the reported cases. Forensic studies indicated that the cause is mainly related to the interaction between these buried structures with the expansion of the clayey soils of Mila basin where large areas have been classified by Athmania et al. [25] as high sensitive to the shrink/swell phenomena (see Figure 1). The Aine-Tine site located in Aine-Tine municipality is crossed by an 800 mm diameter water supply pipeline which carries water to more than 12 municipalities of the Mila province.
The serious economic and environment effects of these damages highlight the importance of taking into consideration the unsaturated behavior of such soils during the design and the construction of pipelines in Algeria. The safety and durability of buried pipeline-systems necessitate careful design studies which need to take into account all of the probable contributing factors related to the materials (i.e., soil and pipeline), environment (i.e., freezing, drying and wetting cycles) and loading (i.e., overburden stresses, traffic) to study the behavior of pipelines usually buried in the vadose zone (i.e., above the water table) at 2 to 3 m from the ground surface in order to predict and reduce the possible damage of the structures.

In this study, a Finite Element Analysis has been conducted using the commercial software SIGMA/W to assess the pipeline structural response expressed in terms of radial induced axial force $F_A$, shear force $F_S$ and bending moment $M_B$ that are exerted on the pipeline perimeter following the expansive soil volume change provoked by a 4 mm/day rainfall infiltration event lasting for 30 days (i.e., 1 month). The present study aims to understand the effect soil wetting on the pipeline/unsaturated-soil interaction by considering (1) different rainfall time durations and (2) the initial soil suction conditions. The water supply pipeline coming from the Beni-Haroun Dam of 800 mm diameter buried at 2 m depth in Aine-Tine (Mila, Algeria) expansive soil was used in this numerical analysis. First, this article begins with an introduction which presents a summary of the literature on previous pipeline-soil interaction studies and the background and goals of the present study followed by a presentation of the methodology used to perform the analysis and then an overview of the SIGMA/W finite element software used in this investigation. Following a detailed description of the main features of the Aine-Tine area the numerical model adopted (geometry and boundary conditions) the material characteristics used to simulate Aine-Tine expansive soil and pipeline structure are exposed. Finally, the results obtained are presented and discussed where conclusions and future works are proposed at the end of this study.

![Expansive clay](image1.jpg)

**Figure 1. Water leakage point at Aine-Tine site**

2. **Research Methodology**

Pipelines are linear structures used for transporting fluids such as gas and liquids. They are usually buried in various types of soils over long distances and can cross through areas with high sensitivity to expansion. In the present work, a two-dimensional numerical simulation is performed to analyse the transverse structural behaviour of a buried pipeline following expansion movements due to rainfall precipitation. The paper is a contribution to help engineers to predict reasonably what are the probable reasons that cause several water leakage points and which constitute continuous disturbance of the water supply process. The main steps of the numerical analysis are summarized in the flowchart bellow (see Figure 2):
3. The Study Area

The study area is located in the Mila province in the North-East of Algeria (see Figure 3) which belongs to the eastern Alpine chain of the north of Algeria. It is north bordered by the Skikda and Jijel Provinces, south by the Batna Province, West by the Setif province and by the Constantine province in the East. It is situated at 490 km from Algiers (i.e., the capital of the country) and at about 60 km from the city of Constantine. Mila covers an area of 3,550 km², composed by five communes (towns) and a number of 32 municipalities. The Aine-Tine site can be bracketed by latitudes 06°18′30″ and 06°19′00″ and longitudes 36°25′30″ and 36°25′45″. Geologically, Mila’s exposure soil consists of Mio-Pliocene and Quaternary continental deposits which covers a set of carbonate, bedrock, of Cretaceous to Eocene ages. More than 70% of the geological formations are Alluviums and clays, where these occupy 50.14% and 21.83%, respectively [25].

Figure 2. Research flowchart of the present paper

Figure 3. Aine-Tine area location
The study area belongs to the semi-arid climate region of the country that receives precipitation below potential evapotranspiration, but not as low as the desert climate. The semi-arid climate is characterized by only two seasons: a hot dry season in summer and a cold rainy season in winter. Detailed investigations of the hydrological properties of the eastern side of Algeria have been presented by Mebarki [26]. The annual average precipitation in Mila province is 678.5 mm/year, even though in 1984 the maximum annual precipitation reached 1058 mm. The maximum monthly average is between 120 and 145 mm/month (i.e., December). During the period October to April, December is the rainiest month of the Mila region. It must be stressed that Mila is a geotechnical problem prone area (i.e., landslides and a shrink/swell movements). Rainfall precipitation is the principal trigger of these phenomena which occur especially during the wet season. This is due to the high sensitivity of clayey soils related to the decrease in suction following precipitations. Site investigation results indicated the water table level has been found to be located at about 15 m depth which is consistent with the data reported in [26].

4. Finite Element Analysis using SIGMA/W

The present study is a two-dimensional plane-strain numerical simulation of the soil-pipeline-environment interaction. The powerful 2D finite element SIGMA/W software was used. SIGMA/W is one of the modules of GEOSTUDIO software suite proposed by GeoSlope International Ltd. [27]. The suite is usually used for modeling stresses and deformations in ground and structural materials. The SIGMA/W module is widely used for geotechnical research and practice by civil and mining communities to carry out slope stability, consolidation, shrinkage/swelling and structural analysis in both saturated and unsaturated conditions ([28–30]). In our case, it is used here to investigate the transverse structural behavior of a steel pipeline buried in a high expansive soil considering the unsaturated behavior of the soil.

Similar 2D software programs (Plaxis 2D, FLAC 2D) can equally be used to carry out the present study. When comparing SIGMA/W and Plaxis 2D, it must be indicated that both give similar and conservative results due to the assumption of plane strain modeling. SIGMA/W is more flexible and faster than Plaxis 2D. Its software library contains linear, nonlinear, elastic and elastoplastic soil constitutive models to simulate soils with different meshing options controlling the form (i.e., triangle, square or combination between both) and the global size with possibility of refining. Plaxis 2D on the other hand, provides five mesh resolutions which can be applied (very coarse, coarse, medium, fine and very fine). As for the boundary conditions and loading options SIGMA/W makes it possible to consider constant values as for the usual x, y fixities or functions (i.e., spline, linear or step function) of time to simulate environment conditions such as precipitation or seismic actions. SIGMA/W gives also the possibility to model structures such as pipelines and retaining walls using bar and beam elements. The partial differential force equilibrium and water continuity equations are the two fundamental equations which govern the mechanical behavior of soil and flow behavior of the water phase in unsaturated soils, respectively [29]. These two equations are incorporated into SIGMA/W, which simplifies the two-dimensional saturated and unsaturated hydromechanical analyses. The unsaturated behavior of soils is governed by an incremental stress-strain relationship developed by Fredlund and Rahardjo [8] where it is a function of the net normal stress \((\sigma - u_a)\) and the suction \((u_a - u_w)\) as stress state variables. Shear strength and stiffness of unsaturated soil are functions of these two stress state variables. The two-dimensional matrix formulation of the stress-strain relationship incorporated in SIGMA/W is as follows:

\[
\begin{bmatrix}
\Delta e_x \\
\Delta e_y \\
\Delta y_{xy}
\end{bmatrix} =
\begin{bmatrix}
1 & -\mu & 0 \\
-\mu & 1 & 0 \\
0 & 0 & 2(1+\mu)
\end{bmatrix}
\begin{bmatrix}
\Delta (\sigma_x - u_a) \\
\Delta (\sigma_y - u_a) \\
\Delta (\tau_{xy})
\end{bmatrix} +
\begin{bmatrix}
1 & 0 & 0 \\
0 & 1 & 0 \\
0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
\Delta (u_a - u_w) \\
\Delta (u_a - u_w) \\
\Delta (u_a - u_w)
\end{bmatrix}
\]

(1)

Where \(e_x, e_y\) and \(y_{xy}\) are the normal and shear strain components, \(\sigma_x, \sigma_y\) and \(\tau_{xy}\) are the normal and shear stress components, \(E\) is the Young’s modulus of the soil structure, \(H\) is the elasticity modulus with respect to a change in suction and \(\mu\) is the Poisson’s ratio. \(H\) is estimated adopting the relationship \(H = E/(1-2\mu)\) according to Vu and Fredlund [14, 31] published works. SIGMA/W helps users to consider automatically the additional shear strength of unsaturated soils evolving from the suction by extending the Mohr-Coulomb failure criterion as shown in Equation 2.

\[
\tau_f = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \Theta \tan \phi'
\]

(2)

Where \(\Theta\) is the normalized volumetric water content VWC or \([(\theta_w - \theta_s)/(\theta_s - \theta_p)]\) where \(\theta_p\) is the volumetric water content at residual state, \(\theta_s\) is the volumetric water content at saturated state. Modeling of the unsaturated behavior requires the definition of the hydraulic property functions of the soil, which is the soil-water-characteristic curve SWCC and the permeability function \(K\) which relates the volumetric water content and the hydraulic conductivity to the soil suction. The calculation of the variation of unsaturated modulus of elasticity \((E_{unsat})\) due to the change in saturation state of the soil is possible using the incorporated function which defines the modulus of elasticity as a function of effective stress or suction (see Section 6.3) which takes into account changes in pore water pressures under precipitation, irrigation activities or rising of GWT. In addition to the van Genuchten [32] model of the SWCC
included in Plaxis 2D, the used software includes the Fredlund and Xing [33] model and allows users to define their own soil water characteristic curve. Additionally, unlike others similar 2D geotechnical software’s (i.e., Plaxis2D or FLAC2D), SIGMA/W allows practitioners to define and combine geometries and analyze multiple geotechnical problems in a single modeling project and to solve analyses in parallel which considerably reduces simulation times.

5. Numerical Model

5.1. Model Geometry

A uniform soil profile is assumed to extend down to 4 m depth under the ground level with a length of 8 m horizontally, as shown in Figure 4. The horizontal and vertical dimensions of the model are set to avoid the effects of boundary conditions. These dimensions are related directly to the pipeline diameter where according to the recommendations presented in many published researches ([21, 22, 34]) it was found after simulations that it is sufficient to numerically evaluate induced large displacements on pipeline transverse cross section by having a cross-section model with 10 and 5 times the pipe diameter for horizontal and vertical dimensions, respectively. On the basis of these recommendations the present study was carried out adopting the model dimensions shown in Figure 4.

The meshes were generated automatically using the available option in SIGMA/W quadrilateral and triangles elements where a unified mesh size of 0.25 m was applied for the soil region. For the pipe, the beam element perimeter is divided into 24 equal segments, which gives an angle increment of 15° for each segment. The length s of each segment can be obtained by applying the following equation $s = r \cdot \theta$ (i.e. 0.104m) where r is the radius of the pipeline and $\theta$ is the angle. The total number of elements in the model is equal to 690 elements.

5.2. Boundary Conditions

The boundary conditions of the model are: left and right boundaries are fixed only in the x-direction and the bottom boundary is fixed both in the x and y directions. To model the rainfall effect, a 4 mm/day constant rainfall unit flux $q$ (mm/day) lasting for 30 days, which corresponds to the minimum daily average occurring in the rainiest month in the Mila basin (i.e., December) was assigned to the top surface of the model. As for the suction, the boundary conditions are given in Figure 5. To study the effect of suction, four initial profiles of suction P1, P2, P3 and P4 are imposed on the model based on the assumption of a hydrostatic linear distribution of the suction above the water table to simplify the study. The imposed suction profiles may correspond to different water table levels or reflect different degrees of surface evapotranspiration that characterize arid and semi-arid climate regions such as Mila province. The theoretical water table levels corresponding to the suction profiles P1, P2, P3 and P4 are equal to 15, 30, 60 and 120 m, respectively. The first profile corresponds to the Aine-Tine area water table depth which gives a suction value at the top surface of the model equal to 147 kPa (see Figure 5). The other profiles are obtained by a double increment way (P1=1/2 P2, P2=1/2 P3 and P3=1/2 P4). Therefore, four pressure head values equal to -11, -26, -56 and -116 m are, respectively, maintained along the bottom boundary nodes of the model for each suction profile during the rainfall simulation.
6. Material Characteristics

6.1. Clay

The pipeline is buried in the vadose zone of the Aine-Tine area which is classified according to USCS classification system as CH, which is an inorganic clay of high plasticity with a clay content equal to 52%. The geotechnical parameters of Aine-Tine clay are presented in Table 1. They were summarized from in situ and laboratory investigation reports of soil studies carried out on the Aine-Tine site between the years 2015 and 2017. The integrated extended elastic-perfectly plastic Mohr-Coulomb (MC) constitutive model was used to model the clayey soil. It is successfully used to analyze soil-pipeline interaction problems considering the unsaturated behavior of the soil [16, 19, 20]. The gravity is set to 9.81 m/s².

| Material | Soil property | Value |
|----------|---------------|-------|
| Aine-Tine Clay | Particle size distribution (%) | Sand=26, Silt=22, Clay=52 |
| | Atterberg limits (%) | w_l=65.32, w_p=27.87, I_p=37.45 |
| | Void ratio e | 0.69 |
| | Total unit weight γ_t (KN/m³) | 19.1 |
| | Dry unit weight γ_d(KN/m³) | 15.7 |
| | Natural moisture content w_nat (%) | 22 |
| | Elastic modulus E_sat (kPa) | 1000 |
| | Poisson’s ratio µ | 0.4 |
| | Angle of internal friction φ (°) | 15 |
| | Cohesion C (kPa) | 22 |
| | Saturated permeability K_sat(m/day) | 5e-3 |
| USCS Classification | CH, Inorganic clay of high plasticity |

6.2. Hydraulic Property Functions, SWCC and K

Physical parameters such as particle size distribution and plasticity indices are the main factors that determine the shape of the soil water characteristic curve (SWCC). An estimation of the SWCC function was carried out based on literature reviews where the similarity of the index properties was considered as the criterion of selection amongst
many published soil results around the world. The estimated SWCC has been found to be close to that of the Regina clay presented in [13, 29, 35] papers. Using Pedo-Transfer prediction options available in SIGMA/W software such as the van Genuchten [32] and Fredlund and Xing’s [33] methods, the SWCC can be estimated. In the present study, the van Genuchten model shown in Equation 3 was used to estimate the SWCC using the particle size distribution data and the plasticity index results shown in Table 1.

\[
S = S_r + \frac{1 - S_r}{\left[1 + \left(\frac{u_a - u_w}{\alpha}\right)^n\right]^m}
\]  

(3)

Where \(S\) is the degree of saturation, \((u_a - u_w)\) is the suction, \(S_r\) is the residual saturation and \(a\), \(n\), and \(m\) are fitting parameters where \(n = 1/(1 - m)\). The corresponding van Genuchten [32] values of these parameters used in this study are as follow: \(a=0.5\), \(n=1.08\). The pore water movement is governed by the soil permeability function \(K\). Using the SWCC and the saturated permeability \(K_{sat}\). The permeability function \(K\) was generated based on the van Genuchten method [32] method which is integrated in the GeoStudio software suite library. \(K_{sat}\) is assumed to be \(5\times10^{-3}\) m/day.

6.3. Modulus of Elasticity Variation with Respect to Suction

Many researchers have reported the dependency between soil stiffness and the suction value \((u_a - u_w)\) [15, 36, 37]. Oh et al. [36] proposed a semi-empirical model to predict the unsaturated Young’s modulus \(E_{unsat}\) of unsaturated cohesionless soils as shown in Equation 4 using the saturated elastic modulus \(E_{sat}\) and two fitting parameters. The extended model by Adem and Vanapalli [38] to cover all types of soils, was utilized in this study.

\[
E_{unsat} = E_{sat} \left[1 + \alpha \frac{(u_a - u_w)}{(P_a/101.3)} (S)\beta\right]
\]  

(4)

Where \(P_a\) is the atmospheric air pressure and \(\alpha\) and \(\beta\) are fitting parameters. In this analysis, the fitting parameters \(\alpha\) and \(\beta\), are taken equal to 2 and 0.1, respectively. The chosen values of \(\alpha\) and \(\beta\) are appropriate for fine-grained expansive soils (i.e., clay) as confirmed by Adem and Vanapalli [38, 39].

6.4. Pipeline

Different diameter sizes of water transport pipelines are buried in Mila Basin. The analyzed section of pipeline buried in Aine-Tine site is 800 mm diameter and has a diameter-to-thickness ratio equal to 40 and assumed to be covered by 2 m clayey soil. The pipeline ring was modeled as a beam element using the linear elastic model. The behavior of the steel pipeline is governed by its rigidity, which is a function of its geometry dimensions and elastic modulus. The elastic modulus is assumed to be 2 GPa which corresponds to a flexible pipeline classification. The Aine-Tine pipeline characteristics are summarized in Table 2. Figure 4 shows the cross section used to calculate the moment of inertia that is essential for transverse pipeline simulations in the case of 2D plane-strain analysis. The interface between the pipeline and the soil was created using line area option. The interface was modeled using the same characteristics of the surrounding clayey soil presented in Table 1.

| Material          | Soil property | Value |
|-------------------|---------------|-------|
| External diameter | \(Q_{ext}\) (mm) | 800   |
|                   | Thickness \(t\) (mm) | 20    |
| Aine-Tine Pipeline| Dr/t          | 40    |
|                   | Young’s modulus \(E\) (GPa) | 2     |

7. Results and Discussion

The numerical analyses were conducted to simulate the effect of the volume changes that occur in the expansive soil subjected to a saturation process acting as an external hydraulic loading (i.e., rainfall precipitation) on the buried pipeline. The results are presented in terms of induced (1) heave and deformations and (2) internal forces that apply along the ring of the pipeline following the decrease of soil suction. The rainfall time duration is taken into account by considering the results of four chosen time durations equal to 4, 8, 15 and 30 days of simulation and four initial suction profiles were studied to consider the effect of the initial suction (i.e., P1, P2, P3 and P4).
7.1. Suction Variations

Rainfall infiltration provides a downward flux which increases the water content of the soil and decreases consequently the suction (i.e. negative pore water pressure) of the soil. In this study, the effect on the suction of the rainfall that occurs during the rainiest month (December) in Mila basin is modelled. For the four initial suction profile conditions P1, P2, P3 and P4, the suction variations were evaluated as transient seepage analysis under 4 mm/day rainfall infiltration using SIGMA/W software where the hydraulic response is governed by the soil water characteristic curve $SWCC$ and the permeability function $K$.

Figure 6 presents the variation of the suction profiles at the outer edges of the model in response to the wetting process throughout the simulation periods. It can be noticed that the soil suction decreases gradually starting from the top surface going to the bottom of the soil depth. And proportionally with time as shown for each chosen time duration of simulation 0, 4, 8, 15 and 30 days. Figure 6a, 6b, 6c and 6d are presented to provide a comparison where it is obvious that the higher the initial suction profile (P1, P2, P3 and P4), the higher the range of suction fluctuations. Similarities of the hydraulic response were obtained in many published studies [14, 31, 40].

Figure 6. Evolution of soil suction with rainfall time for the suction profiles P1, P2, P3 and P4

7.2. Soil Volume Changes (Heave)

The induced decrease of suction discussed in the above section leads to volume changes associated with deformations within the soil. Figure 7 depicts the contours of vertical displacements, for the whole model, after 30 days of 4mm/days rainfall intensity for different initial suction profile conditions P1, P2, P3 and P4 presented, respectively, in Figure 7a, 7b, 7c and 7d. The direction and magnitude of soil deformations can be viewed by the red arrow vectors for which the ascending vertical direction dominates the movements. The final calculated displacements at the top surface of the model, crown and invert of the pipeline are illustrated in Figure 8 where those obtained at the top surface are equal to 1.54, 2.01, 3.81 and 8.51 cm which correspond to the suction profiles P1, P2, P3 and P4, respectively, while those calculated at the crown of the pipeline are as follow 0.54, 0.82, 1.80 and 4.24 cm. Besides, matching proportionality between heave and decrease of soil suction have been obtained by Rajeev and Kodikara [18] during their experimental and numerical investigations on the effect of swell movements on buried pipelines following the moisture content increase to saturation level within the soil due to a succession of capillary rises. Figures 7a, 7b, 7c and 7d are presented to provide a comparison at the end of the simulation (30 days) where it can be observed that the higher the initial suction the higher the induced heave at the upper surface of the model. The magnitudes of heave decrease with depth where the ratios of the vertical displacements at the crown of the pipeline with those at the top surface of the model are equal to 35, 41, 47 and 50% which correspond to P1, P2, P3 and P4, respectively.
Figure 7. Vector and maximum displacements at the crown of the pipeline and at the top surface of the model for different initial suction after 30 days of rainfall infiltration.

Figure 8. Y-Displacement values at the Invert, Crown and Top surface for P1, P2, P3 and P4.

Figure 9 shows the distortions in the soil regions (with 5 x times magnification) around the pipeline area when the simulation period has reached the 30th day of rainfall infiltration for each initial soil suction profile. The deformed meshes indicate the magnitude and direction of the ground movements while the black ring indicates the original location of the pipe. Based on the comparison between the four deformed meshes (from P1 to P4), it is clear that deformations around the pipeline perimeter increase as the initial suction increases because of the increasing role of the soil suction that acts like a hydraulic stress that produce soil deformations which can be clearly seen in Equation 5 which correspond to the second member of the right hand side of the stress-strain relationship presented in Equation 2.
where, \( \Delta x \) (suction), \( \Delta y \) (suction) and \( \Delta xy \) (suction) are the normal and shear strain components with respect to the soil suction. Figures 9c and 9d showed that the soil at the pipeline crown and invert deformed significantly due to the high state of unsaturation in the suction P3 and P4, respectively, which induces a vertical displacement of the pipe. In the case of P1 and P2 the deformed meshes are presented in Figures 9a and 9b, respectively, for which the observed deformations mainly concern the crown of the pipeline due the limited expansion occurred at the pipeline invert mainly due to the low state of unsaturation of these two initial conditions.

A cross-section ovalization of the pipeline can be clearly observed in Figures 9b, 9c and 9d because the final vertical displacements (obtained at the 30th days) at the crown of pipeline are higher than those of the invert which leads to an increase of the pipeline diameter in the vertical direction as can be observed from Figure 8 where the increase of the diameter in the vertical direction reaches 1 and 2.1 cm taking into account the profile P3 and P4, respectively (GWT= 60 and 120m). This is obviously associated with a decrease in diameter at the springline reaching a value of 2.04 cm in the case of the profile P4 (GWT=120m). Such behavior which is consistent with the findings of [18] and [23] can be attributed firstly to (1) the vertical direction of seepage as the upper zones saturate before the lower zones which is principally linked to the magnitude of suction with depth, SWCC and K of the soil and secondly to (2) the volume change magnitudes which is proportional to the range of suction fluctuations. The same observations were made by Adem and Vanapalli [13] and Vu and Fredlund [14]. The profile P1, P2, P3 and P4 can represent different degrees of evapotranspiration during a dry season. As a result of this, it can be concluded from here that the higher the evapotranspiration or the depth of the water table the higher the induced distortions within the expansive soil mass and consequently the induced loading on the pipeline ring.

![Figure 9. Deformed mesh around the pipeline perimeter (x5 magnitude)](image)

### 7.3. Radial Forces

The obtained radial forces in the present analysis will be discussed considering those obtained in some previous studies carried out under other external loading such as unsupported excavation and frost heave in cold regions.

- Al-Khazaali et al. [20] used successfully SIGMA/W to investigate numerically the effect of a succession of unsupported excavations on nearby buried water transport pipeline considering of the effect of suction.
- Huang et al. [17] studied experimentally the circumferential induced strains and bending following vertical displacement due to frost heave in a permafrost.

#### 7.3.1. Axial Forces \( F_A \)

The volume changes that occur in the expansive soil mass associated with heave at the top surface will result in stresses and strains on the pipeline ring and this will consequently lead to additional loads expressed in this study as pipeline internal forces \( F_A, F_S \) and \( M_B \).

The result of the axial forces \( F_A \) along the perimeter of the Aine-Tine pipeline under different rainfall infiltration time durations at the chosen time steps 4, 8, 15, and 30 days considered for the four initial suction profiles P1, P2, P3 and P4 are shown in Figure 10. It can be seen that positive values are obtained at the crown and invert (at angles 0° and 180°) positions of the pipeline perimeter which means the development of axial compression forces for all the initial suction profiles while negative values appeared in the springline (right, 90° and left, 270°) which means axial tension forces were developed in this location of the perimeter. Figure 10d shows the developed axial forces after 30 days of rainfall for the suction profile P4 which corresponds to the deepest GWT used in this study. The tension forces reached 67.26 kN and 66.03 kN at the crown and invert, respectively. The obtained values are 38% higher than those obtained after 15 days of simulation (Figure 10c) while the increase reaches 104% between 8 days (Figure 10b) and 15 days. Regarding the compressive axial forces, the magnitudes are slightly lower and the ratios are equal to 14% and 139% from 8days to 15 days and from 15 days to 30 days, respectively. Due to the downward vertical direction of...
infiltration flow, the obtained values at the crown location are slightly higher than those obtained in the invert while they are identical at the left and right springline locations. The axial compression and tension forces proportionally increased with both the duration of the infiltration from 4 to 30 days and the initial soil suction profiles from P1 to P4. From the results of Figure 10, it can be concluded that for a shallow water table which corresponds to the results indicated by the red contours and a short period of rainfall, as in the case of 4 days (Figure 10a), the pipe ring will receive negligible axial forces.

The presented results reflect the behavior of the soil-pipeline system with respect to the rain time duration and the initial suction conditions. This can be ascribed to the soil suction contribution to the unsaturated behavior of the Aine-Tine soil which is expressed as volume expansion. The decrease of the suction (see Figure 6) around the pipeline ring will produce stresses and strains at all nodes of the model following the rainfall infiltration. This will consequently, lead to an increase in tension axial forces at the crown and invert location. In contrast, the decrease of the soil suction with rainfall for all the cases P1, P2, P3 and P4 will also cause an increase in compression axial forces at the springline location. Similar observations have been reported by Alkhazaali et al. [20] however, the axial compressive forces were obtained at the springline location (i.e., 90° and 270°) while tensile forces were obtained at the crown and invert locations (i.e., 0° and 180°). This is due to the dominance of the horizontal displacements induced by a succession of unsupported vertical excavation operations unlike in the present study, the vertical displacements are dominant due to the expansion of the Aine-Tine soil upwards.

Figure 10. Axial force $F_A$ distribution along pipeline perimeter (kN) with time

7.3.2. Shear Forces $F_S$

Figure 11 presents the variation of the circumferential shear forces $F_S$ along the cross section pipeline wall. The shear forces developed on the pipeline wall increase with the rainfall time duration. Where the maximum peak (Positive) values were calculated approximately at angles 30° and 225° while the minimum peak (Negative) values were calculated at angles 135° and 330° while those obtained by Al-Khazaali et al. [20] are located approximately in the same locations (Approximately ± 5°) with opposite signs and increase with the depth of excavation which has the same role as the downward flux which induces the vertical movements in the present analysis. Furthermore, the obtained shear forces have symmetrical distribution whereas those obtained by Al-Khazaali et al. [20] are higher in
magnitude and much higher at the side close to the front of the excavation. This is due to the horizontal direction of the large sliding movements as discussed above. It can be noticed from each of the plotted curves of Figures 11a, 11b, 11c and 11d, that obviously the induced shear forces increase with respect to the initial suction profile \((u_a - u_w)\) conditions P1, P2, P3 and P4, which corresponds to different theoretical water table levels or may reflect different degrees of evapotranspiration which are very probable in arid and semi-arid climate such as the site of the present study. The magnitudes of the calculated shear forces are lower than those of the axial forces where as shown in Figures 11a, 11b, 11c and 11d, the positive peak values for P4 (with purple color) calculated at the location 30° are equal to 0.25, 1.11, 1.92 and 2.32 kN and those calculated at 225° reached 0.04, 0.41, 1.4 and 1.95 kN. The same observation about the change of magnitudes can be drawn regarding the influence of the duration and the depth of the GWT as for the axial forces.

![Figure 11. Shear force distribution along pipeline perimeter (kN) with time](image)

7.3.3. Bending Moment \(M_B\)

The bending moment results \(M_B\) calculated along the pipeline perimeter are shown in Figure 12 using the color red, blue, green and purple to represent the results of P1, P2, P3 and P4, respectively. It is clear that they present a similar trend and present a consistent response as for the previous internal forces (axial and shear forces). The peak positive values were found at angles 90° and 270° while, for the peak negative values, they were found at angles 0° and 180° which are the similar location for the peak axial force values but with different signs. Regardless of the time duration chosen it can be observed that peak values of calculated bending moment \(M_B\) increased almost twice between each two subsequent initial suction condition from P1 to P4 and have dissymmetric distribution (in Figures 12a, 12b, 12c, and 12d). Similar results in terms of trends but with different magnitudes are reported in [17] for buried gas pipelines in cold regions which experienced vertical movements caused by differential frost heaves leading to the generation of circumferential strains as well as bending moments which are worse in the transition zone between frozen and unfrozen soil. Based on the comparison between the results of the peak values plotted in Figure 12, it was found that regardless of the duration of the simulation or the depth of the GWT, the order with respect to the magnitudes is as follows: the crown (0°), springline (90° and 270°) and the invert (180°) while the maximum values of bending moment are calculated using the profile P4 after 30 days of rainfall and they reach values, respectively, 0.51, 0.41 and
0.38 kN.m. Reading attentively the Figure 11 and Figure 12, it can be seen that at zero shear stresses location, the peak values (positive and negative) of bending moments were obtained. From the obtained results, it obvious that the internal forces are sensitive to the variation of the soil suction following a rainfall event on high expansive soil such as the clay of Aine-Tine site.

![Figure 12. Bending moment $M_B$ distribution along pipeline perimeter (kN) with time](image)

8. Conclusions

In the present study, numerical analyses were performed to investigate the transverse structural behavior of buried pipelines in expansive soils considering the unsaturated behavior of soil under the effect of 4mm/day rainfall precipitation lasting for 30 days as an external hydraulic loading. Considering the water supply 800 mm pipeline coming from the Beni-Haroun Dam and buried at 2 m depth in Aine-Tine (Mila, Algeria) high expansive soil, four simulations were performed to demonstrate the effect of the initial suction profiles P1, P2, P3 and P4 which may represent different degrees of aridity that characterize regions with arid and semi-arid climate.

The results obtained were expressed in terms of (1) changes in soil suction, (2) heave and deformations and (3) radial forces (Axial $F_A$, Shear $F_S$ forces and Bending moment $M_B$). Analysis of the results allowed to reach the following conclusions:

- Rainfall events can cause additional loads on buried pipelines in expansive soils because of the important role of the unsaturated behavior of such soils especially in arid and semi-arid regions.
- Due to the soil saturation process caused by the progressive rainfall infiltration water within the soil, the expansive soil exhibits volume changes associated with heave at the ground surfaces following decrease in suction indicating that the effect of the soil suction ($u_a - u_w$) must be taken carefully into consideration, especially where the seasonal variation of moisture content is very high.
- The induced soil heave and the radial forces exerted on the pipeline wall (Axial, Shear and Bending moment) increase with the initial soil suction (P1, P2, P3 and P4) and also with the rainfall duration (4, 8, 15 and 30 days).
The initial suction values worked out from the water table depth or from the degree of evapotranspiration and the importance of the rainfall precipitation are important factors that impact seriously the unsaturated behavior of the soil behavior as well as the response of the pipeline structure.

The pipeline cross-section ovalization can be the logic reason to many water leakage points along the paths of the Mila water supply pipelines including the one located in Aine-Tine site especially near the pipeline joints.

The results of the present study are very useful to understand the origin of the damages reported on the Ain-Tine water supply pipeline. The results of this study made clear that in order to achieve a realistic design of buried lifeline infrastructures, the unsaturated behavior of the embedding expansive soil should be taken into consideration under realistic climatic conditions of the region. Moreover, this study analyzed only the transverse behavior of the pipeline structure under simplified hypotheses. In order to achieve more realistic results, a representative daily precipitation data (i.e., intensity, duration and frequency) would be used instead. In addition, many types of the pipelines such as rigid, flexible, welded and jointed and with different diameter are buried to transport fluids and which need to be considered in the future studies. It would be also interesting to study the longitudinal behavior of the pipeline.

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11. Conflicts of Interest

The authors declare no conflict of interest.

12. References

[1] Zhang, Xiong, and Jean-Louis Briaud. “Three Dimensional Numerical Simulation of Residential Building on Shrink-Swell Soils in Response to Climatic Conditions.” International Journal for Numerical and Analytical Methods in Geomechanics 39, no. 13 (March 23, 2015): 1369–1409. doi:10.1002/nag.2360.

[2] Ozer, Mustafa, Resat Ulusay, and Nihat Sinan Isik. “Evaluation of Damage to Light Structures Erected on a Fill Material Rich in Expansive Soil.” Bulletin of Engineering Geology and the Environment 71, no. 1 (August 30, 2011): 21–36. doi:10.1007/s10064-011-0395-2.

[3] Yılmaz, Işık. “The Effect of Swelling Clays on a Water Transport Canal Between Köklüce HPP and Erbaa HPP (Turkey).” Bulletin of Engineering Geology and the Environment 66, no. 4 (February 24, 2007): 467–472. doi:10.1007/s10064-007-0086-1.

[4] Clark, Curtis M. “EXPANSIVE-SOIL EFFECT ON BURIED PIPE.” Journal - American Water Works Association 63, no. 7 (July 1971): 424–427. doi:10.1002/j.1551-8833.1971.tb04116.x.

[5] Robert, D. J., K. Soga, and T. D. O’Rourke. “Pipelines Subjected to Fault Movement in Dry and Unsaturated Soils.” International Journal of Geomechanics 16, no. 5 (October 2016). doi:10.1061/(asce)gm.1943-5622.0000548.

[6] Uzundurukan, Soner, Siddika Nilay Keskin, Hüseyin Yıldırım, Turan Selçuk Göksan, and Ömür Çimen. “Suction and Swell Characteristics of Compacted Clayey Soils.” Arabian Journal for Science and Engineering 39, no. 2 (November 7, 2013): 747–752. doi:10.1007/s13369-013-0852-2.

[7] Ito, Maki, Shahid Azam, and Yafei Hu. “A Two Stage Model for Moisture-Induced Deformations in Expansive Soils.” Environmental Systems Research 3, no. 1 (June 24, 2014). doi:10.1186/s40068-014-0019-5.

[8] D. G. Fredlund et H. Rahardjo, Soil Mechanics for Unsaturated Soils. John Wiley & Sons, 1993.

[9] Biot, Maurice A. “General Theory of Three-Dimensional Consolidation.” Journal of Applied Physics 12, no. 2 (February 1941): 155 – 164. doi:10.1063/1.1712886.

[10] Fredlund, D. G., and N. R. Morgenstern. “Constitutive Relations for Volume Change in Unsaturated Soils.” Canadian Geotechnical Journal 13, no. 3 (August 1, 1976): 261–276. doi:10.1139/t76-029.
[11] Kumar, Sanjeev, Anil Kumar Saha, and Sanjeev Naval. “Influence of Jute Fibre on CBR Value of Expansive Soil.” Civil Engineering Journal 6, no. 6 (June 1, 2020): 1180–1194. doi:10.28991/cej-2020-03091539.

[12] Han, Zhong, Sai K. Vanapalli, and Zehra Nil Kutlu. “Modeling Behavior of Friction Pile in Compacted Glacial Till.” International Journal of Geomechanics 16, no. 6 (December 2016). doi:10.1061/(asce)gm.1943-5622.0000659.

[13] Adem, H H, and S K Vanapalli. “Constitutive Modeling Approach for Estimating 1-Dheave with Respect to Time for Expansive Soils.” International Journal of Geotechnical Engineering 7, no. 2 (April 2013): 199–204. doi:10.1179/1938636213z.00000000024.

[14] Vu, Hung Q, and Delwyn G Fredlund. “Challenges to Modelling Heave in Expansive Soils.” Canadian Geotechnical Journal 43, no. 12 (December 1, 2006): 1249–1272. doi:10.1139/t06-073.

[15] Randeniya, Chamal, D.J. Robert, Chun-Qing Li, and Jayantha Kodikara. “Large-Scale Experimental Evaluation of Soil Saturation Effect on Behaviour of Buried Pipes Under Operational Loads.” Canadian Geotechnical Journal 57, no. 2 (February 2020): 205–220. doi:10.1139/cgj-2018-0544.

[16] Robert, D. J., K. Soga, T. D. O’Rourke, and T. Sakanoue. “Lateral Load-Displacement Behavior of Pipelines in Unsaturated Sands.” Journal of Geotechnical and Geoenvironmental Engineering 142, no. 11 (November 2016): 04016060. doi:10.1061/(asce)gt.1943-5660.0001504.

[17] Huang, Scott L., Kun Yang, Satoshi Akagawa, Masami Fukuda, and Shunjie Kanie. “Frost Heave Induced Pipe Strain of an Experimental Chilled Gas Pipeline.” Innovative Materials and Design for Sustainable Transportation Infrastructure (June 2015). doi:10.1061/9780784479278.037.

[18] Rajeev, Pathmanathan, and Jayantha Kodikara. “Numerical Analysis of an Experimental Pipe Buried in Swelling Soil.” Computers and Geotechnics 38, no. 7 (November 2011): 897–904. doi:10.1016/j.compgeom.2011.06.005.

[19] Robert, Dilan, and Kenichi Soga. “Soil-Pipeline Interaction in Unsaturated Soils.” Mechanics of Unsaturated Geomaterials (March 7, 2013): 303–325. doi:10.1002/9781118616871.ch13.

[20] Al-Khazaali, Mohammed, Sai K. Vanapalli, and Won Taek Oh. “Numerical Investigation of Soil–pipeline System Behavior Nearby Unsupported Excavation in Saturated and Unsaturated Glacial Till.” Canadian Geotechnical Journal 56, no. 1 (January 2019): 69–88. doi:10.1139/cgj-2017-0411.

[21] Vazouras, Polynikis, Panos Dakoulas, and Spyros A. Karamanos. “Pipe–soil Interaction and Pipeline Performance Under Strike–slip Fault Movements.” Soil Dynamics and Earthquake Engineering 72 (May 2015): 48–65. doi:10.1016/j.soildyn.2015.01.014.

[22] Oghabi, Mohsen, Mehdi Khoshvatan, and Aminaton Marto. “Evaluation of the Response of Buried Steel Pipelines Subjected to the Strike-Slip Fault Displacement.” Civil Engineering Journal 3, no. 9 (October 7, 2017): 661–671. doi:10.21859/cej-03093.

[23] Saadeldin, Ramy, Yafei Hu, and Amr Henri. “Numerical Analysis of Buried Pipes Under Field Geo-Environmental Conditions.” International Journal of Geo-Engineering 6, no. 1 (June 26, 2015). doi:10.1186/s40703-015-0005-4.

[24] Wahid. Chettah, “Investigation des propriétés minéralogiques et géomécaniques des terrains en mouvement dans la ville de Mila « Nord-Est d’Algérie »», Thèse de Magistère, University of Batna 1 Hadj Lakhdar, Batna, Algérie, 2009.

[25] Athmania, Djamel, Abdelkader Benaissa, Achour Hammadi, and Mounir Bouassida. “Clay and Marl Formation Susceptibility in Mila Province, Algeria.” Geotechnical and Geological Engineering 28, no. 6 (June 23, 2010): 805–813. doi:10.1007/s10706-010-9341-5.

[26] Azzedine. Mekbarki. “HYDROLOGIE DES BASSINS DE L’EST ALGERIEN: RESSOURCES EN EAU, AMENAGEMENT, ET ENVIRONNEMENT” (2005 thèse de doctorat d’état, University of Mentouri, Constantine.

[27] GeoSlope International Ltd, Sigma/W user’s guide for stress-deformation analysis. GEO-SLOPE International Ltd, Calgary, AB, Canada, 2007.

[28] Al-Khazaali, Mohammed, and Sai K. Vanapalli. “A Novel Experimental Technique to Investigate Soil–Pipeline Interaction Under Axial Loading in Saturated and Unsaturated Sands.” Geotechnical Testing Journal 43, no. 1 (March 15, 2019): 20180059. doi:10.1520/gtj20180059.

[29] Qi, Shunchao, and Sai K. Vanapalli. “Hydro-Mechanical Coupling Effect on Surficial Layer Stability of Unsaturated Expansive Soil Slopes.” Computers and Geotechnics 70 (October 2015): 68–82. doi:10.1016/j.compgeo.2015.07.006.

[30] Bahrami Balf, Teimouri, Atanaz, and Ahad Bagherzadeh Khalkhali. “Stability Control of Narmab Dam and Sensitivity Analysis of Reliability Coefficients.” Civil Engineering Journal 4, no. 9 (September 30, 2018): 2197. doi:10.28991/cej-03091150.
[31] Vu, Hung Q, and Delwyn G Fredlund. “The Prediction of One-, Two-, and Three-Dimensional Heave in Expansive Soils.” Canadian Geotechnical Journal 41, no. 4 (August 1, 2004): 713–737. doi:10.1139/t04-023.

[32] van Genuchten, M. Th. “A Closed-Form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils.” Soil Science Society of America Journal 44, no. 5 (September 1980): 892–898. doi:10.2136/sssaj1980.03615995004400050002x.

[33] Fredlund, D.G., and Anqing Xing. “Equations for the Soil-Water Characteristic Curve.” Canadian Geotechnical Journal 31, no. 4 (August 1, 1994): 521–532. doi:10.1139/t94-061.

[34] Vazouras, Polynikis, Spyros A. Karamanos, and Panos Dakoulas. “Finite Element Analysis of Buried Steel Pipelines Under Strike-Slip Fault Displacements.” Soil Dynamics and Earthquake Engineering 30, no. 11 (November 2010): 1361–1376. doi:10.1016/j.soildyn.2010.06.011.

[35] Azam, Shahid, Imran Shah, Mavinakere E. Raghunandan, and Maki Ito. “Study on Swelling Properties of an Expansive Soil Deposit in Saskatchewan, Canada.” Bulletin of Engineering Geology and the Environment 72, no. 1 (January 6, 2013): 25–35. doi:10.1007/s10064-012-0457-0.

[36] Oh, Won Taek, Sai K. Vanapalli, and Anand J. Puppala. “Semi-Empirical Model for the Prediction of Modulus of Elasticity for Unsaturated Soils.” Canadian Geotechnical Journal 46, no. 8 (August 2009): 903–914. doi:10.1139/t09-030.

[37] Zhang, Junhui, Junhui Peng, Jue Li, and Jianlong Zheng. “Variation of Resilient Modulus with Soil Suction for Cohesive Soils in South China.” International Journal of Civil Engineering 16, no. 12 (May 30, 2018): 1655–1667. doi:10.1007/s40999-018-0315-y.

[38] Adem, Hana H., and Sai K. Vanapalli. “Prediction of the Modulus of Elasticity of Compacted Unsaturated Expansive Soils.” International Journal of Geotechnical Engineering 9, no. 2 (March 18, 2014): 163–175. doi:10.1179/1939787914y.0000000050.

[39] Adem, Hana H., and Sai K. Vanapalli. “Elasticity Moduli of Expansive Soils from Dimensional Analysis.” Geotechnical Research 1, no. 2 (June 2014): 60–72. doi:10.1680/gr.14.00006.

[40] Qi, Shunchao, and Sai K. Vanapalli. “Influence of Swelling Behavior on the Stability of an Infinite Unsaturated Expansive Soil Slope.” Computers and Geotechnics 76 (June 2016): 154–169. doi:10.1016/j.compgeo.2016.02.018.
Shrinkage Behavior of Conventional and Nonconventional Concrete: A Review

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Abstract

Concrete is indeed one of the most consumed construction materials all over the world. In spite of that, its behavior towards absolute volume change is still faced with uncertainties in terms of chemical and physical reactions at different stages of its life span, starting from the early time of hydration process, which depends on various factors including water/cement ratio, concrete proportioning and surrounding environmental conditions. This interest in understanding and defining the different types of shrinkage and the factors impacting each one is driven by the importance of these volumetric variations in determining the concrete permeability, which ultimately controls its durability. Many studies have shown that the total prevention of concrete from undergoing shrinkage is impractical. However, different practices have been used to control various types of shrinkage in concrete and limit its magnitude. This paper provides a detailed review of the major and latest findings regarding concrete shrinkage types, influencing parameters, and their impacts on concrete properties. Also, it discusses the efficiency of the available chemical and mineral admixtures in controlling the shrinkage of concrete.

Keywords: Shrinkage; Autogenous Shrinkage; Plastic Shrinkage; Crack; Conventional Concrete; Nonconventional Concrete.

1. Introduction

Through its lifespan, concrete undergoes several physical and chemical changes, which normally led to shrinkage of concrete, especially at an early age, when the initial hydration processes take place [1]. The shrinkage of concrete at an early stage of hardening may lead to the initial formation of cracks that vary in shape and size and depends on the concrete constituents and surrounding conditions, including temperature and/or the moisture state that may lead to volumetric deformation [2, 3]. Shrinkage cracking starts to form while the concrete is still in the plastic state and continues through the hardened state due to the applied stress on concrete particles. These stresses are created as a result of the consumption of the mixing water existed within the cement paste, which takes place after losing the water available within the pores [4]. The shrinkage and formation of cracks within the concrete texture are nearly inevitable. Generally, cracks occur when the tensile stress in brittle material exceeds its rapture strength [5]. However, the interaction of the factors and parameters affecting the development and propagation of cracks in concrete makes it difficult to isolate the effect of each parameter alone [6]. The main nonlinear phenomena that govern the shrinkage behavior of concrete at early-age may include the evolution of stiffness properties, development of thermal strains, creep, and cracks formation [7].

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These concerns in the making conventional concrete led not only to higher demand in producing nonconventional concretes with higher performance in terms of strength but also to a more durable mixture that provides a better shrinkage resistance. Currently, many studies, numerically or and experimentally, in recent years have focused on the shrinkage behavior of concrete and the governing factors impacting different types of shrinkage, including chemical shrinkage, plastic shrinkage, drying shrinkage, carbonation shrinkage, and thermal shrinkage. The findings of these studies showed that the shrinkage properties are affected by the environmental conditions, aggregate type, cementitious materials, water/cement ratio, and chemical admixtures. Some cementitious replacements, such as fly ash, proved to improve the resistance of different types of concrete against shrinkage and development of cracks [8-10]. The use of recycled materials and polymeric waste has become a trend due to environmental consideration. Different shrinkage reducing admixtures are used in concrete to enhance its shrinkage properties. With all these factors on shrinkage behavior, it is necessary to provide a detailed review paper that collects and discusses the recent findings in the literature. Thus, this paper is intended to summarize and compare previous investigations on the shrinkage behavior of concrete.

### Table 1. Research methodology

| Databases Searched                          | Google Scholar, Scopus, Science Direct, Web of science |
|--------------------------------------------|--------------------------------------------------------|
| Some keywords used                         | Shrinkage, cracking of concrete, conventional, and nonconventional concrete. |
| Year range                                 | 1988–2020                                              |
| Language                                   | English                                                |
| Types of publications covered              | Original research articles, conference papers, review papers, reports |
| Criteria used to produce the preliminary database for initial review | • Most publications showing under the search results, using the above selection criteria, up to the last page of the search index, were downloaded. |
| Criteria used to shortlist the final database for a detailed, comprehensive review that is presented in this manuscript | • A database was created in excel using the following: paper title, year published, shrinkage type, remarks of the study. |

### 2. Types of Shrinkage

Various types of shrinkage can occur over the mixture’s maturity process and life span, which are classified as autogenous, plastic, drying, carbonation, and thermal.

#### 2.1. Autogenous Shrinkage

Autogenous shrinkage is mainly caused by the chemical contraction of cement, which takes place during the hydration process due to the consumption of the internal moisture content from the concrete subsurface [11]. ACI 116R describes the autogenous shrinkage as the “change in volume caused by continuous cement hydration, excluding the effects of applied load and change in either thermal condition or moisture content” [12]. Autogenous shrinkage cracking usually appears in the first few hours after concrete casting as it starts to harden. The autogenous shrinkage increases with decreasing water/cement (w/c) ratio [13-15]. Therefore, it is a concerning issue in high-strength and/or high-performance concrete mixtures [1, 3]. In concrete with a higher w/c, there can be a form of early cracking wherein the cracks are located directly over the steel [16]. The autogenous shrinkage strain needs to be controlled in the first 24 hr to avoid reaching a high value and subsequent premature cracks. The crucial period through which the highest tensile stresses occurred was recorded is after 24 to 48 hr from the setting time, and thus, proper autogenous shrinkage prevention is needed [17]. In earlier studies, autogenous shrinkage was not the main concern as it was noted to occur only at a meager w/c ratio away from the allowed practical ranges for standard concrete mixes at that time. However, usage of admixtures, as superplasticizers or silica fume, the concrete resistance to such a shrinkage noticed to be reduced [1]. On the other hand, increasing the aggregate content in the mixture reduces its autogenous shrinkage, as a result of the decrease in the total volume of the cement paste and the high volume stability of aggregate [13].

#### 2.2. Plastic Shrinkage

This type refers to the shrinkage that occurs while the concrete is still in its plastic state, and the concrete constituents are weakly bonded. It happens when the rate of losing the mixture’s water is higher than the rate of bleeding at the surface before the final setting [18, 19]. As the surface of concrete dries, a complex menisci process shapes the liquid between the particles at and near the surface, and as a result of capillary action; a tensile capillary pressure within the liquid phase is created, which eventually cause the development of plastic shrinkage as the pressure increases [19]. The content of cementitious materials used in a concrete mix, w/c ratio, and the surrounding environment are the main parameters that control plastic shrinkage cracking. The incorporation of higher fine materials within the mixture, such as silica fume, slag, or fly ash, will delay the final set of concrete and make the
cracking more likely to happen.

In the case of hot weather, low humidity, and high-speed wind, plastic shrinkage is highly expected [20, 21]. Its cracks usually take a linear shape, with no definite pattern and range from a few centimeters to a few meters in length. Typically, they weaken the concrete structure and allow the diffusion of moisture and other aggressive species into the concrete mass unless they are very shallow and narrow, which eventually cause the formation of reinforcement corrosion and lead to concrete failure [21]. Reinforcing concrete with fibers reduces the width of the cracks developed by plastic shrinkage where the higher the fiber volume, the smaller the width of the crack is achieved, in spite this additive increases the rate and magnitude of evaporation [22].

2.3. Drying Shrinkage

Similarly, this type describes the loss of water from evaporation; it usually starts in the first weeks and continues for months to develop [23]. The ambient conditions are the main factor affecting the rate of drying shrinkages, such as the relative humidity and temperature surrounding the concrete. Furthermore, the concrete characteristics, including w/c ratio, cement type, aggregate, and incorporating admixtures influence the value of drying shrinkage significantly [11]. Hence, it is recommended to reduce the w/c ratio in the mixture to a minimum amount with the usage of superplasticizer, which can decrease the intensity of drying shrinkage [9, 10]. Other measures to improve the behavior against drying shrinkage can be done by utilizing low heat cement or shrinkage cement. It can be achieved by modifying the w/c ratio and reducing the fine materials to the amount that can just produce adequate workability and finishing characteristics to prolong the time of moisture curing, setting contraction joints (temporary or permanent). Another technique is by using chemical admixtures such as a water reducer or shrinkage reducing admixtures. On the other hand, cracking can be controlled by good workmanship, proper proportioning of the mixture, and sufficient jointing performed soon after hardening [16].

2.4. Carbonation Shrinkage

The carbonation shrinkage is known to have the lesser severe consequences of carbonation on cementitious composites. It reorganizes the concrete microstructure, reduces concrete volume, declines paradoxical in the porosity, and varies differential shrinkage between the surface and the bulk of concrete [24]. Besides, drying and autogenous shrinkage, the concrete is also subjected to volume reductions due to the carbonation reactions [1]. These changes can increase the carbonation level, due to the formation of surface cracks, and thus reduce the initiation time preceding the corrosion of the steel reinforcement and is more severe for highly porous cementitious materials [24].

2.5. Thermal Shrinkage

The thermal shrinkage is swelling and contraction of concrete due to the stresses developed by the heat generated from the hydration process and the changes in temperature between the concrete and the surrounding conditions. This concrete heating process is also called thermal expansion in the early stage after casting and typically occurs in the first 12 hours. In general, if an even temperature with a gradient exists along the cross-section of concrete while it starts to cool down and reaches thermal equilibrium between its layers, a thermal strain is formed. This strain may spread cracks depending on the elasticity of thermal expansion and the size of the concrete section [3]. At later stages, thermal shrinkage will be developed due to the fluctuation in the concrete surface temperature and the ambient one, which could vary significantly between the inner and outer surface of the concrete section. Often, control joints are used to limit the thermal cracking developed by thermal expansions and volume change. Thermal stress is affected by many factors, including w/c ratio, aggregate type, a daily variation of temperature during construction, location of the structure, size of the structural members, construction quality. In addition, the formworks being used has a little impact on thermal stress [25].

3. Impacts of Aggregate Type on Shrinkage Behavior

Actually, as the content of coarse aggregate in concrete increases, its autogenous and drying shrinkage decrease at a given age [26]. Through the literature, various types of aggregate were used in a concreting application, which influences the shrinkage and cracking properties of concrete. This section discusses the findings of previous researches on the influence of aggregate type on the shrinkage of concrete.

3.1. Lightweight Aggregate Concrete

The aggregates with over bulk density than the common aggregates are defined as lightweight aggregates, and they can be produced from gasification of slag and/or fly ash with or without the addition of other waste materials. Lightweight aggregate can produce concretes with high compressive strength for different uses. It enhances the shrinkage properties as studies shown, the autogenous shrinkage is reduced when lightweight aggregates are used in concrete. Lightweight aggregates have a porous texture that tends to absorb and store water, and this water provides a
moisture source for the internal curing processes after concrete casting [17, 27, 28]. Using presoaked lightweight aggregate was reported to change the autogenous deformation type in the concrete from shrinkage to swelling [29]. In other respects, lightweight aggregate concrete showed less drying shrinkage in comparison to normal aggregate concrete due to the small shrinkage that lightweight aggregate exhibits [30, 31].

3.2. Recycled Aggregate Concrete

The use of recycled materials has become a trend in all sectors and the less the construction sector, and the use of recycled aggregates saves natural aggregates resources, including stones and rocks. The term recycled aggregate usually refers to demolition waste. However, the use of the recycled aggregates has an impact on the shrinkage properties of concrete as, adding more recycled aggregate instead of the natural one to concrete increases its drying shrinkage [32]. It was noted that recycled aggregate concrete reaches almost its full shrinkage within the first 28 days, considering the conservative ambient conditions and keeps increasing moderately up to 60 days, after which it starts to balance progressively. While under standard curing conditions, it shrinks in a fluctuation manner. Throughout the entire age, samples did not show considerable shrinkage. As the ambient temperature rises, the shrinkage increases at low relative humidity [33]. The shrinkage strain at 28 days of recycled coarse concrete is higher than the conventional aggregate concrete [34, 35]. Recycled aggregate concrete with a replacement of 20% had similar shrinkage to normal concrete at an early age and a 4% higher within the first 6 months. A 50% replacement resulted in around 10% higher shrinkage than that in normal concrete, while 100% replacement caused about 70% greater shrinkage compared to the control mixture concrete for the first six months [35, 36]. The inclusion of superplasticizer had the best effect on the carbonation depth, which is the most important property that is affected by construction and demolition waste recycled aggregate. In contrast, the chloride ingress had the lowest impact. The composition of the recycled aggregates affects the shrinkage more than the size of the aggregate fraction replaced. Superplasticizers does not enhance the shrinkage resistance of concrete with recycled aggregates sourced from construction and demolition wastes as that of recycled aggregates sourced from concrete [37]. In general, the curing age plays a key role in controlling the shrinkage of recycled aggregate concrete.

3.3. Polymeric Aggregate Concrete

Polystyrene aggregate is used in the production of lightweight concrete, which has been increasingly used in the construction world, considering that polystyrene aggregates produce low density concretes. Polystyrene aggregates showed higher shrinkage rates compared to plain concrete at an early age, and with the shrinkage increases as polystyrene aggregate content increases [38]. Similarly, the use of tire aggregate in concrete showed a higher shrinkage in comparison to normal reinforced concrete, and this can be attributed to the high w/c ratios of tire aggregate and to the low capacity of tire aggregate in restraining the shrinkage of the cement paste. The increase in aggregate size due to the use of tire aggregate increased the shrinkage [39]. Furthermore, using aggregate submerged in polymer latex or water repellant after drying it can decrease the shrinkage by almost 13% [42].

3.4. Other Types of Aggregate

For other types of aggregates produced from rocks and stones used in concrete, the shrinkage properties could vary depending on the condition and sizes. For instance, using shale instead of quartzite aggregate in concrete reduces its shrinkage. However, as the ratio of shale (either in sand or stone form) in aggregate gets higher, the long-term drying shrinkage increases proportionally [40]. Limestone aggregate concrete exhibited better performance against drying shrinkage compared to sandstone aggregate concrete due to the change in volume of the tested aggregates [41]. Granite rock is highly affected by its initial moisture condition, where initially wet granite rock showed higher ultimate shrinkage with shorter shrinkage half-time. Initially, dry granite rock showed lower ultimate shrinkage but longer shrinkage half-time. The addition of pre-dried aggregate can lower the shrinkage to almost 7%. Furthermore, using aggregate submerged in polymer latex or water repellant after drying it can decrease the shrinkage by almost 13% [42]. Concrete prepared with tailing sand and tailing stone in class 30 or class 60 exhibited lower drying shrinkage compared to natural sand and common crushed limestone. However, the drying shrinkage of concrete of class 60 was higher than that in class 30, regardless of whether tailing sand, tailing stone, natural sand, or common crushed limestone is used [43].

4. Influence of Supplementary Cementitious Materials on Shrinkage of Concrete

Ternary cementitious components, including ground granulated blast furnace slag and pozzolanic materials, are widely used in construction as a partial replacement of cement. These materials are also known to increase the mechanical and durable performance of concrete. However, alkali-activated slag concrete experienced higher total shrinkage than conventional concrete [44]. Due to the high chemical shrinkage, fine pore structure, and particle shape of blast furnace slag, the autogenous shrinkage of granulated blast furnace slag concrete increases with rising replacement level of blast furnace slag at the same w/c compared to plain concrete [45]. Under appropriate curing
conditions, the shrinkage of alkali-activated slag concrete is slightly reduced with shrinkage reducing agents [44]. Also, the use of blast furnace slag coarse aggregate instead of normal coarse aggregate in alkali-activated slag concrete was found to decrease the drying shrinkage remarkably [44]. Furthermore, the replacement of slag up to 15% increased the drying shrinkage. However, the use of a higher percentage resulted in decreasing the shrinkage [46-49].

While for concrete with 60% slag, cement replacement by volume showed a 35% reduction in shrinkage at 30 days and a 19% reduction in shrinkage at one year under 14 days of curing in comparison to plain cement mixes. Using slag cement with porous limestone coarse aggregate with water internally cured showed a more significant decrease in free shrinkage in comparison to plain cement mixes that have low absorption coarse aggregate [47]. Adding silica fume to concrete increased the plastic and drying shrinkage strain. Furthermore, silica fume cement concrete showed a 69% increment in plastic shrinkage than conventional concrete. Whereas, short-term shrinkage was higher in silica fume cement concrete, and the long-term was somewhat similar in both silica fume cement concrete and normal concrete [48, 50]. As concluded by Zhang et al. under 65% relative humidity and seven days curing, a w/c ratio in the range of 0.26 to 0.35 and a silica fume in the range of 0% to 10% of cement by mass, exhibited a higher total shrinkage due to the greater autogenous shrinkage with no effect on drying shrinkage. Reducing the w/c ratio from 0.35 to 0.30 in ordinary concrete with 0% silica fume resulted in a significant increase in autogenous shrinkage from 40 to 180 micro strains at 98 days. While in the case of concrete with silica fume up to 10% by the weight of cement, the autogenous shrinkage up to 98 days, Figure 1, was relatively high even at a w/c ratio of 0.35 (>200 micro strains) [14].

![Figure 1. Effect of silica fume on the autogenous shrinkage of concrete [14]](image)

The use of other supplementary materials such as fly ash shows a positive impact on concrete shrinkage behavior, whereas the content of fly ash increases, the drying shrinkage decreases [51]. A fly ash replacement beyond 10% will reduce the shrinkage considerably [52]. Using fly ash of 50% replacement or less resulted in decreasing the free drying shrinkage [9, 52]. The effect of fly ash in concrete has a higher on the drying shrinkage when wet curing time is reduced compared to ground granulated blast furnace slag and normal concrete [49]. Using fly ash and slag in concrete enhances the workability and reduces the plastic shrinkage area using 20% of fly ash and slag [53]. Micro silica and slag cement exhibited less drying shrinkage compared to the fly ash cement mixture [54]. Fly ash and iron ore tailing powder in concrete showed similar behavior in terms of autogenous shrinkage. As the replacement increased, the autogenous shrinkage reduced proportionally at the same water/binder [55]. Some other cement replacement fine materials can affect the shrinking property of concrete, such as using limestone powder in concrete showed less or equal drying shrinkage conventional cement concrete [56]. The concrete with 10% limestone powder showed moderately less drying shrinkage compared to traditional concrete of cement, while for 20% and 30% of limestone powder, drying shrinkage was less than in conventional cement concrete. This result is due to that drying shrinkage reduces as the water/binder ratio reduces and that as the cement content replaced by limestone powder increases, lesser hydration products and so smaller drying shrinkage [56, 57]. The addition of limestone powder to concrete decreases the autogenous shrinkage [58]. Replacement of cement paste by fine limestone exhibited a reduction in ultimate shrinkage strain by almost 28% [59]. Also, drying shrinkage of the concrete with 10% limestone powder is slightly smaller than that of plain cement concrete at 5 years, while that containing 20 and 30% limestone powder showed lower drying shrinkage as shown in Figure 2.
Belferrag et al. [60] studied the effects of using river sand instead of dune sand on the drying shrinkage of concrete. They found that the concrete mixture containing 40% dune sand and 60% of river sand provides a remarkably enhanced performance than using 100% dune sand in concrete, as shown in Figure 3. The replacement of cement by metakaolin from 5% to 20%, decrease the drying shrinkage of concrete considerably. However, beyond 20% replacement, the effect of metakaolin was reversed, and shrinkage increased [61, 62]. Furthermore, using 100% glass cullet sand in concrete resulted in a 16% decrease of shrinkage, which can be attributed to near-zero porosity and water absorption characteristics by limiting the moisture movement from concrete to surroundings [63].

5. Shrinkage Behavior of Various Concrete Mixtures

5.1. Plain Concrete with Admixtures

A study by Montani [64] has shown that using shrinkage reducing admixtures in plain concrete can result in decreasing the drying shrinkage by about 30%. Combining shrinkage reducing admixtures with pulverized fuel ash or slag leads to a reduced early age autogenous shrinkage [65]. Indeed, different water reducers were tested in terms of their influence on the resistance to plastic shrinkage, and all types of water reducer exhibited satisfactory performance. The polycarboxylate water reducer provided the best performance in comparison to wood calcium and naphthalene [66].

5.2. Self-Compacted Concrete

Previously, Craeye et al. [67] have studied the effect of w/c ratio on the autogenous shrinkage of self-compacting concrete. They concluded that using medium or high w/c ratio shows a great autogenous shrinkage on the first day. In addition, replacement of the total cement with limestone filler exhibits remarkable autogenous shrinkage. While increasing the paste, volume resulted in more shrinkage susceptibility, with only drying shrinkage affecting the paste and very low autogenous shrinkage. It was found that the restrained shrinkage increases along with the probability of cracking as the mineral admixture increases in the concrete mixture [68].

5.3. High-Strength Concrete

For concrete with higher strength, plastic shrinkage is the central consideration. Mora-Ruacho et al. [69] have studied the plastic shrinkage of both normal and high strength concretes and found that the cracking due to plastic
shrinkage is significant in high strength concrete and happens as fast as the exposure of surface starts. They concluded that the width of cracks reduces dramatically with the inclusion of hooked-ended steel fibers. Sicard et al. [70] reported that the drying shrinkage was reduced considerably by applying high levels of compressive stress to the specimen. Utilizing a w/c below 0.30 and adding 10% silica fume in the high-performance concrete mixture was found to prevent the carbonation shrinkage from taking place [71]. Adding some admixtures for high strength concrete has shown a positive impact on controlling types of shrinkage. The use of shrinkage reducing admixtures such as (polyyoxyalkylene alkyl ether) effectively lowers the measured drying and plastic shrinkage value [8, 72]. As well, the autogenous shrinkage of high strength concrete was dropped using superabsorbent polymer. Whereas, incorporation of ground granulated blast furnace slag to internally cured high strength concrete exhibited higher autogenous shrinkage and increased with the increase of slag content [73].

5.4. Fiber Reinforced Concrete

Fibers are usually used in concrete to control cracking, and studies showed that fiber-reinforced mixes exhibited lower early age autogenous shrinkage compared to the control one. The early age autogenous shrinkage deformations decreased by using fiber of 0.38% by volume. Polypropylene fibers showed better behavior than hooked-end steel fibers in terms of early-age autogenous shrinkage. Fibers used in white cement concrete had a slight effect in terms of early-age autogenous shrinkage in comparison with other cement mixes, which have shown better effect [74]. Adding steel fibers decreases the free shrinkage of the conventional concrete beam by up to 30% for controlled curing conditions while it decreases the shrinkage at the lower half of the beam by about 30% in the case of uncontrolled curing conditions [75]. The drying shrinkage was reduced by the granulometric correction of dune sand and the inclusion of fibers. In general, the effect of introducing fibers on the shrinkage strain is inconstant. Adding fibers to concrete has a slight effect on its shrinkage if a meager aspect ratio of the fibers is used in the mixture [76]. The high-volume fraction of fibers reduces the drying shrinkage in concrete [60]. The drying and plastic shrinkage can be reduced by 1/3 when 0.4% polypropylene fibers are incorporated in the concrete mixture. The increase of fiber volume fraction decreases the total crack area, number of cracks, and maximum crack width.[77-79]. The inclusion of 0.3% of flax fiber (percentage by volume) showed a reduction in total cracks as 99.5% and 98.5% of maximum crack width in comparison to normal concrete. No considerable effect of fiber length was noticed on unrestrained plastic shrinkage strain or the cracking behavior [80]. The basalt fiber concrete with fiber ratio of 0.1% and length to diameter ratio between 800 and fiber ratio of 0.1% exhibited the best performance against plastic shrinkage and drying shrinkage [81]. Using polyethylene terephthalate fibers up to 0.25% by volume decrease the plastic shrinkage while using more than 0.25% has a slight effect on reducing the plastic shrinkage [82]. Moreover, the use of carbon nanotubes in concrete resulted in decreasing early shrinkage by 54% and long-term shrinkage by 15%. The low w/c ratio and high carbon nanotubes content led to the lowest reduction in the total shrinkage value [83].

5.5. Superabsorbent Polymer Concrete

The use of superabsorbent polymers (SAPs) has many unique characteristics and utilized widely in several applications, not only concrete technology, where it is used to improve the performance and long-term durability of concrete. Studies have shown that superabsorbent polymers impacted the shrinkage properties of concrete, especially with deducted internal curing water. Using pre-absorbed superabsorbent polymer resulted in decreasing the total shrinkage of concrete significantly. Furthermore, the inclusion of a superabsorbent polymer enhanced the carbonation resistance dramatically. While the pre-absorbed superabsorbent polymer deducted internal curing water from mixing water is added, the carbonation resistance improved considerably [84]. The autogenous shrinkage was decreased with the addition of superabsorbent polymer [85-87].

5.6. Recycled Polymers and Rubberized Concrete

Due to its environmental impact, the use of recycled wastes in construction has increased, but the use of some recycled polymers such as polyurethane wastes, which is a polymer composed of organic units joined by carbamate, has some negative impacts as drying shrinkage seems to increase when polyurethane waste content increase. However, with the use of other recycled polymers like polyvinyl chloride, aggregate drying shrinkage is reduced as the polyvinyl chloride volume increase, and when 15% of polyvinyl chloride aggregate were used, 50% of drying shrinkage was decreased [82]. Moreover, the addition of styrene-butadiene polymer in concrete showed no plastic shrinkage cracking. Furthermore, styrene-butadiene polymeric is used to resolve the shrinkage cracking caused by the rapid hydration heat produced by rapid hardening cement as it exhibited small length changes due to the low w/c ratio used. Besides, it has undergone lower shrinkage levels, effectively reduced cracking, as concluded by Won et al. [88]. For rubberized concrete, which is usually used in applications such as highways also in buildings as an earthquake shockwave absorber by mixing recycled rubber crumb with concrete. The drying shrinkage of rubberized concrete was found to be influenced by the number of rubber particles in the concrete mixture with the more the rubber replacement ratio, the higher the measured drying shrinkage is expected. This observation is similar to the general behavior of
rubberized concrete in which the rubber contents have a negative influence on the mechanical properties of the produced concrete [89, 90]. These reductions are mainly attributed to the fact that rubber aggregates are weaker and more flexible than the natural ones [91]. On the other hand, pre-treating the rubber particles by NaOH solution resulted in a reduction in drying shrinkage [92, 93]. In addition, when less than 4 mm crumb rubber aggregate is used, the strain capacity is improved, resulting in increasing free shrinkage. This is attributed to the low stiffness aggregate introduced by rubber particles, which means less internal restraint and thus results in a more considerable change in the length due to the increase in shrinkage. Self-compacting rubberized concrete was found to provide better performance in terms of shrinkage as compared to the control mixture. Utilizing waste rubber particles was observed to improve the deformability of the mixture under pre-failure loads and to result in a lesser shrinkage [94]. The use of styrene-butadiene rubber with the copolymer (ethylene-vinyl acetate) in concrete resulted in a reduction of drying shrinkage as the polymer-cement ratio is increased [95].

5.7. Geopolymer Concrete

Geopolymer concrete is a new type of concrete that has zero Portland cement content. Geopolymer concrete can utilize several abandoned materials to create a strong binder that has superior properties as compared to ordinary cement concrete [96-98]. Geopolymer concrete uses an alkaline solution as an activating solution that largely influences its properties, including shrinkage [99, 100]. The drying shrinkage of fly ash-based geopolymer concrete exhibits deficient value as compared to ordinary concrete [101]. This fact occurs from the smaller pores size of the geopolymer binder, which tends to increase the resistance to diffusion, and a longer time is required for drying [102]. Contrary to this, some researchers have reported that the drying shrinkage of slag-based geopolymer concrete was higher than ordinary concrete [103]. Indeed, the shrinkage behavior of geopolymer concrete is not a function of the aluminosilicate source only but also the properties of the alkaline solution used. It was reported that the higher concentration of the sodium hydroxide solution resulted in a higher autogenous shrinkage while it reduced the drying shrinkage values. Sodium silicate content showed less effect on the drying and autogenous shrinkage values [104].

6. Conclusion

This review paper summarizes the findings of more than 100 research studies published over the last 30 years on different types of concrete and the factors influencing the shrinkage properties of concrete. In general, several studies showed that shrinkage in concrete starts at an early age. It goes through different stages over time, depending on the shrinkage properties of concrete that are affected by many factors, including the humidity, w/c ratio, type of coarse aggregate, shape, and hardness of aggregates, use of supplementary cementitious materials, and curing conditions. Shrinkage cracking is closely related to water loss in the paste, where the drying condition and limiting the amount of paste is controlled by w/c ratio (0.4-0.5), a proper curing technique, and chemical admixtures. Whereas, for higher strength concrete when w/c is around 0.3, plastic shrinkage is the primary concern as well as autogenous shrinkage and the use of shrinkage reducing admixtures such as (polyoxalkylene alkyl ether). On the other hand, fiber reinforcement is considered as a useful technique for reducing and limiting the development of plastic shrinkage since it enhances the tensile creep and delays the cracking of concrete. The aggregates that are used in the production of concrete have different impacts on shrinkage properties based not only on the type of the aggregates but also on its shape and hardness. Besides, larger-sized aggregate absorbs lower water and hence shows a reduced shrinkage. Moreover, recycled aggregate from polymers as polystyrene and tire rubbers revealed a negative effect and increased the plastic shrinkage. In contrast, polymeric binders such as polyvinyl chloride aggregate, styrene-butadiene polymer, and geopolymer binders enhanced concrete characteristics and prevented shrinkage. The usage of superabsorbent polymers improved the performance and long-term durability of concrete. Notably, the pre-absorbed superabsorbent polymer that was decreased the total shrinkage of concrete. Another important factor that affects the shrinkage properties of concrete is the inclusion of supplementary cementitious materials such as fly ash, slag, and limestone powder. Generally, the short-term shrinkage was observed to be higher in the case of concrete with silica fume. In contrast, the higher the percentage of replacement of ordinary cement with limestone powder provides better results in resisting the long-term shrinkage.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References

[1] Holt, E. "Contribution of mixture design to chemical and autogenous shrinkage of concrete at early ages." Cement and Concrete Research 35 (2005): 464-72. doi:10.1016/j.cemconres.2004.05.009.

[2] Al-Gburi, M. Restraint effects in early age concrete structures. Luleå University of Technology, 2015, doi: 10.2749/101686614x14043795570570.
[3] Holt, E. E. Early age autogenous shrinkage of concrete. Finland: Technical Research Centre of Finland, 2001, doi:10.1201/9781482272123-23.

[4] Thelandersson, S., A. Mårtensson and O. Dahlbom. "Tension softening and cracking in drying concrete." Materials and Structures 21 (1988): 416-24. doi:10.1007/bf02472321.

[5] Al Houri, A., A. Habib, A. Elzokra and M. Habib. "Tensile testing of soils: History, equipment and methodologies." Civil Engineering Journal 6 (2020): 591-601. doi:10.28991/cej-2020-03091494.

[6] Gilbert, R. I. "Cracking caused by early-age deformation of concrete-prediction and control." Presented at Procedia Engineering, 2017. Elsevier Ltd, 172, 13-22. doi:10.1016/j.proeng.2017.02.012.

[7] Sofi, M., P. Mendis and D. Bawjea. Early age concrete thermal and creep effects: Relevance to anchorage zones of post-tensioned members protective structures view project ductility design of very-high strength reinforced concrete (vhsc) (100-50 mpa) columns view project. 2008, doi:10.1201/b10571-46.

[8] Hatami, B., A. M. Ramezanianpour and A. S. Daryan. "Investigation on the effect of shrinkage reducing admixtures on shrinkage and durability of high-performance concrete." Journal of Testing and Evaluation 46 (2017): 141-50. doi:10.1520/jte20170055.

[9] Hu, X., Z. Shi, C. Shi, Z. Wu, B. Tong, Z. Ou and G. de Schutter. "Drying shrinkage and cracking resistance of concrete made with ternary cementitious components." Construction and Building Materials 149 (2017): 406-15. doi:10.1016/j.conbuildmat.2017.05.113.

[10] Jasiczak, J., P. Szymański and P. Nowotarski. "Wider perspective of testing early shrinkage of concrete modified with admixtures in changeable w/c ratio as innovative solution in civil engineering." Procedia Engineering 122 (2015): 310-19. doi:10.1016/j.proeng.2015.10.041.

[11] Jianxia, S. "Durability design of concrete hydropower structures." In Comprehensive renewable energy. S. Jianxia. 6. Elsevier Ltd, 2012, 377-403. DOI:10.1016/b978-0-08-078772-0.00019-3.

[12] ACL. Aci 116r cement and concrete terminology. American Concrete Institute, 2000.

[13] Tazawa, E.-I. and S. Miyazawa. Experimental, study on mechanism of autogenous shrinkage of concrete. 1995, 1633-38.

[14] Zhang, M. H., C. T. Tam and M. P. Leow. "Effect of water-to-cementitious materials ratio and silica fume on the autogenous shrinkage of concrete." Cement and Concrete Research 33 (2003): 1687-94. doi:10.1016/s0008-8846(03)00149-2.

[15] Kohno, K., T. Okamoto, Y. Isikawa, T. Sibata and H. Mori. Effects of artificial lightweight aggregate on autogenous shrinkage of concrete. 1999, 611-14. doi:10.1016/s0008-8846(98)00202-6.

[16] Walker, H. N., L. Stephen and P. E. Stutrmman. Petrographic methods of examining hardened concrete: A petrographic manual. 2006.

[17] Cusson, D. and T. Hoogeveen. "Internal curing of high-performance concrete with pre-soaked fine lightweight aggregate for prevention of autogenous shrinkage cracking." Cement and Concrete Research 38 (2008): 757-65. doi:10.1016/j.cemconres.2008.02.001.

[18] Kuhlman, L. A. Cracks in lmc overlays; how do they get there; how serious are they; what to do about them. Washington, DC: 1991.

[19] Cabrera, J. G., A. R. Cusens and Y. Brookes-Wang. Effect of superplasticizers on the plastic shrinkage of concrete. 1992, 149-55. doi:10.1680/macr.1992.44.160.149.

[20] ACL. Aci 224r-01 control of cracking in concrete structures. American Concrete Institute, 2001, doi:10.14359/10632.

[21] Almusallam, A. A., M. Maslehuddin, M. Abdul-Waris and M. M. Khan. Effect of mix proportions on plastic shrinkage cracking of concrete in hot environments. 1998, 353358. doi:10.1016/s0950-0618(98)00019-1.

[22] Qi, C., J. Weiss and J. Olek. "Characterization of plastic shrinkage cracking in fiber reinforced concrete using image analysis and a modified weibull function." Materials and Structures 36 (2003): 386-95. doi:10.1007/bf02481064.

[23] Rossi, P. and P. Acker. "A new approach to the basic creep and relaxation of concrete." Cement and Concrete Research 18 (1988): 799-803. doi:10.1016/0008-8846(88)90105-6.

[24] Houst, Y. F. "Carbonation shrinkage of hydrated cement paste." Presented at 4th CANMET/ACI International Conference on Durability of Concrete, Ottawa, Canada, 1997. 481-91.

[25] Zhai, X., Y. Wang and H. Wang. "Thermal stress analysis of concrete wall of lng tank during construction period." Materials and Structures 49 (2015): 1-14. doi:10.1617/s11527-015-0656-9.
[26] Zhang, J., Y. D. Han and Y. Gao. "Effects of water-binder ratio and coarse aggregate content on interior humidity, autogenous shrinkage, and drying shrinkage of concrete." Journal of Materials in Civil Engineering 26 (2013): 184-89. doi:10.1061/(asce)cm.1943-5533.0000799.

[27] Jiajun, Y., H. U. Shuguang, W. Fazhou, Z. Yufei and L. Zhichao. "Effect of pre-wetted light-weight aggregate on internal relative humidity and autogenous shrinkage of concrete." Journal of Wuhan University of Technology-Mater. Sci. Ed. 21 (2006): 134-37. doi:10.1007/bf02861491.

[28] Nabil M. Al-Akhras, M. J. S. A. B. M. "Evaluation of shear-deficient lightweight rc beams retrofitted with adhesively bonded CFRP sheets." European Journal of Environmental and Civil Engineering 20 (2016): 899-913. doi:10.1080/19648189.2015.1084383.

[29] Zielinski, A., M. Kaszynska, S. Skibicki and N. Olczyk. "Development of autogenous shrinkage deformation and strength parameters in self-consolidating concrete with light and natural aggregate." Presented at IOP Conference Series: Materials Science and Engineering, 2019. Institute of Physics Publishing, 471, doi:10.1088/1757-899x/471/3/032019.

[30] Fujiwara, T. "Effect of aggregate on drying shrinkage of concrete." Journal of Advanced Concrete Technology 6 (2008): 31-44. doi:10.3151/jact.6.3.31.

[31] B Malikw, A., J. Aladwan and M. Al-Salaheen. "Agricultural palm oil wastes for development of structural lightweight concrete." International Journal of Civil Engineering and Technology 10 (2019).

[32] Gómez Soberón, J. M. V. "Shrinkage of concrete with replacement of aggregate with recycled concrete aggregate." Presented at 5th International Conference of the American-Concrete-Institute, Cancun, MEXICO, 2002. 209, 475-95. doi:10.1680/sscc.31777.0057.

[33] Liu, Q. and X. N. Zhang. "Experimental study on shrinkage of concrete with recycle crushed brick coarse aggregate." In Applied Mechanics and Materials 438 (2013): 141-44. doi:10.4028/www.scientific.net/amm.438-439.141.

[34] Santos, J. R., F. A. Branco and J. de Brito. "Compressive strength, modulus of elasticity and drying shrinkage of concrete with coarse recycled concrete." In XXX IAHS–World Congress on Housing Housing Construction- An Interdisciplinary Task (2002): 1685-91.

[35] Domingo-Cabo, A., C. Lázaro, F. López-Gayarre, M. A. Serrano-López, P. Serna and J. O. Castaño-Tabares. "Creep and shrinkage of recycled aggregate concrete." Construction and Building Materials 23 (2009): 2545-53. doi:10.1016/j.conbuildmat.2009.02.018.

[36] Soomro, F. A., B. A. Memon, M. Oad, A. H. Buller and Z. A. Tunio. "Shrinkage of concrete panels made with recyclable concrete aggregates." Engineering, Technology & Applied Science Research 9 (2019): 4027-29.

[37] Bravo, M., J. de Brito, L. Evangelista and J. Pacheco. "Durability and shrinkage of concrete with cdw as recycled aggregates: Benefits from superplasticizer's incorporation and influence of cdw composition." Construction and Building Materials 168 (2018): 818-30. doi:10.1016/j.conbuildmat.2018.02.176.

[38] Tang, W. C., Y. Lo and A. Nadeem. "Mechanical and drying shrinkage properties of structural-graded polystyrene aggregate concrete." Cement and Concrete Composites 30 (2008): 403-09. doi:10.1016/j.cemconcomp.2008.01.002.

[39] Bravo, M. and J. De Brito. "Concrete made with used tyre aggregate: Durability-related performance." Journal of Cleaner Production 25 (2012): 42-50. doi:10.1016/j.jclepro.2011.11.066.

[40] Ballim, Y. The effect of shale in quartzite aggregate on the creep and shrinkage of concrete-a comparison with rilem model b3. 2000, 235-42. doi:10.1007/bf02479333.

[41] Yagi, S., C. Aquino, M. Inoue and T. Okamoto. "Volume change of limestone and its effects on drying shrinkage of concrete." In Advanced Materials Research 168 (2011): 738-41. doi:10.4028/www.scientific.net/amr.168-170.738.

[42] Kwan, A. K. H., W. W. S. Fung and H. H. C. Wong. "Reducing drying shrinkage of concrete by treatment of aggregate." Magazine of Concrete research 62 (2010): 435-42. doi:10.1680/macr.2010.62.6.435.

[43] Feng, X. X., Xi, X. L., J. W. Cai, H. J. Chai and Y. Z. Song. "Investigation of drying shrinkage of concrete prepared with iron mine tailings." In Key Engineering Materials 477 (2011): 37-41. doi:10.4028/www.scientific.net/kem.477.37.

[44] Collins, F. and J. G. Sanjayan. "Cracking tendency of alkali-activated slag concrete subjected to restrained shrinkage." Cement and Concrete Research 30 (2000): 791-98. doi:10.1016/s0008-8846(00)00243-x.

[45] Lee, K. M., H. K. Lee, S. H. Lee and G. Y. Kim. "Autogenous shrinkage of concrete containing granulated blast-furnace slag." Cement and Concrete Research 36 (2006): 1279-85. doi:10.1016/j.cemconres.2006.01.005.

[46] Wang, T. C., Q. M. Peng, W. L. Liu and L. F. Feng. "Study on shrinkage of concrete based on bp neural network." In Advanced Materials Research 163 (2011): 3249-57. doi:10.4028/www.scientific.net/amm.163-167.3249.
[47] Yuan, J., W. D. Lindquist, D. Darwin and J. Browning. "Effect of slag cement on drying shrinkage of concrete." American Concrete Institute 112 (2015): 267-76. DOI:10.14359/51687129.

[48] Liu, J. and D. Wang. "Influence of steel slag-silica fume composite mineral admixture on the properties of concrete." Powder Technology 320 (2017): 230-38. doi:10.1016/j.powtec.2017.07.052.

[49] Yang, J., Q. Wang and Y. Zhou. "Influence of curing time on the drying shrinkage of concretes with different binders and water-to-binder ratios." Advances in Materials Science and Engineering (2017): doi:10.1155/2017/2695435.

[50] Ghodousi, P., M. H. Afshar, H. Ketabchi and E. Rasa. Study of early-age creep and shrinkage of concrete containing iranian pozzolans: An experimental comparative study. 2009.

[51] Fauzi, A., M. F. Nuruddin, A. B. Malkawi and M. M. A. B. Abdullah. "Study of fly ash characterization as a cementitious material." Presented at Procedia Engineering. 2016. 148. doi:10.1016/j.proeng.2016.06.535.

[52] Raut, M. V. and S. V. Deo. Use of high volume fly ash on early age shrinkage in concrete. 2018, 2036-46.

[53] Wu, X., H. Li, Q. Li and Z. Wan. "Experimental study on the influences of fly-ash and slag on the plastic shrinkage of concrete." Presented at 6th International Conference on Information Engineering for Mechanics and Materials (ICIMM), Huhhot, PEOPLES R CHINA, 2016. 97, 271-76. doi:10.2991/icimm-16.2016.53.

[54] Mokarem, D. W., R. E. Weyers and D. S. Lane. "Development of a shrinkage performance specifications and prediction model analysis for supplemental cementious material concrete mixtures." Cement and Concrete Research 35 (2005): 918-25. doi:10.1016/j.cemconres.2004.09.013.

[55] Wu, P., C. Wang, Y. Zhang, L. Chen, W. Qian, Z. Liu, C. Jin and L. Li. "Properties of cementious composites containing active/inter mineral admixtures." Polish Journal of Environmental Studies 27 (2018): 1323-30. doi:10.15224/pjoes/76503.

[56] Wang, Q., J. Yang and H. Chen. "Long-term properties of concrete containing limestone powder." Materials and Structures 50 (2017): doi:10.1617/s11527-017-1040-8.

[57] Li, F. L. and Q. Zhu. "Drying shrinkage of concrete affected by content of stone powder in proto-machine-made sand." In Advanced Materials Research 152 (2011): 1176-79. doi:10.4028/www.scientific.net/amr.152-153.1176.

[58] Wang, B., Y. P. Ding, J. Gao, J. F. Sun, M. M. Cui and L. Han. "Effect of limestone powder on autogenous shrinkage of concrete." In Applied Mechanics and Materials 88 (2011): 767-71. doi:10.4028/www.scientific.net/amm.88-89.767.

[59] Kwan, A. K., M. McKinley and J. J. Chen. "Adding limestone fines as cement paste replacement to reduce shrinkage of concrete." Magazine of Concrete Research 65 (2013): 942-50. doi:10.1680/macr.13.00028.

[60] Belferrag, A., A. Kriker, S. Abboudi and S. T. Bi. "Effect of granulometric correction of dune sand and pneumatic waste metal fibers on shrinkage of concrete in arid climates." Journal of Cleaner Production 112 (2016): 3048-56. doi:10.1016/j.jclepro.2015.11.007.

[61] Saand, A. M. A. Keerio and D. K. Bangwar. "Effect of metakaolin developed from local natural material soorh on workability, compressive strength, ultrasonic pulse velocity and drying shrinkage of concrete." Architecture, Civil Engineering, Environment 10 (2017): 115-22. doi:10.21307/acee-2017-025.

[62] Brooks, J. J. and M. M. Johari. "Effect of metakaolin on creep and shrinkage of concrete." Cement and concrete composites 23 (2001): 495-502. doi:10.1016/s0958-9465(00)00095-0.

[63] Dhir, R. K., J. d. Brito, G. S. Ghataora and C. Q. Lye. "Use of glass cullet as a sand component." In Sustainable construction materials. R. K. Dhir, J. d. Brito, G. S. Ghataora and C. Q. Lye. Elsevier, 2018. 167-229. doi:10.1016/b978-0-08-100984-0.00005-9.

[64] Montani, S. "Possibilities to control concrete shrinkage by means of chemical admixtures." CHIMIA International Journal for Chemistry 52 (1998): 208-11.

[65] Qian, X. Q., T. Meng, S. L. Zhan, M. H. Fang and K. L. Qian. "Influence of shrinkage reduce agent on early age autogenous shrinkage of concrete." In Key Engineering Materials 302 (2006): 211-17. doi:10.4028/www.scientific.net/kem.302-303.211.

[66] Qin, H. G., Z. H. Fei, W. Guo and Q. Tian. "The effects of water-reducer on early-age plastic shrinkage of concrete." In Applied Mechanics and Materials 174 (2012): 1113-18. doi:10.4028/www.scientific.net/amm.174-177.1113.

[67] Craeye, B., G. De Schutter, B. Desmet, J. Vantomme, G. Heirman, L. Vandewalle, Ø. Cizer, S. Aggoun and E. H. Kadri. "Effect of mineral filler type on autogenous shrinkage of self-compacting concrete." Cement and Concrete Research 40 (2010): 908-13. doi:10.1016/j.cemconres.2010.01.014.

[68] Rozière, E., S. Granger, P. Turcya and A. Loukili. "Influence of paste volume on shrinkage cracking and fracture properties of self-compacting concrete." Cement and Concrete Composites 29 (2007): 626-36. doi:10.1016/j.cemconcomp.2007.03.010
[69] Mora-Ruacho, J., R. Gettu, C. Olazabal, M. Martin and A. Aguado. "Effect of the incorporation of fiber on the plastic shrinkage of concrete." Presented at 5th RILEM Symposium on Fibre-Reinforced Concretes (FRC), LYON, FRANCE, 2000. 15, 705-14.

[70] Sicard, V., R. Francois, E. Ringot and G. r. Pons. "Influence of creep and shrinkage on cracking in high strength concrete." Cement and Concrete Research 22 (1992): 159-68. doi:10.1016/0008-8846(92)90146-m.

[71] Persson, B. "Eight-year exploration of shrinkage in high-performance concrete." Cement and Concrete Research 32 (2002): 1229-37. doi:10.1016/s0008-8846(02)00764-0.

[72] Shaaban, I. G., M. Saidani, M. F. Nuruddin, A. Malkawi and T. Mustafa. "Serviceability behavior of normal and high-strength reinforced concrete t-beams." European Journal of materials Science and Engineering (2018).

[73] Shen, D., C. Liu, Z. Feng, S. Zhu and C. Liang. "Influence of ground granulated blast furnace slag on the early-age anti-cracking property of internally cured concrete." Construction and Building Materials 223 (2019): 233-43. doi:10.1016/j.conbuildmat.2019.06.149.

[74] Ullah, F., F. Al-Neshawy and J. Punkki. "Early age autogenous shrinkage of fibre reinforced concrete." Nordic Concrete Research 59 (2018): 59-72. doi:10.2478/ncre-2018-0015.

[75] Alekrish, A. A. and S. H. Alsayed. "Shrinkage of fibre and reinforced fibre concrete beams in hot-dry climate." Cement and Concrete Composites 16 (1994): 299-307. doi:10.1016/0958-9465(94)90042-6.

[76] Barr, B., S. B. Hoseinian and M. A. Beygi. "Shrinkage of concrete stored in natural environments." Cement and Concrete Composites 25 (2003): 19-29. doi:10.1016/s0958-9465(01)00044-0.

[77] Banthia, N. and R. Gupta. "Influence of polypropylene fiber geometry on plastic shrinkage cracking in concrete." Cement and Concrete Research 36 (2006): 1263-67. doi:10.1016/j.cemconres.2006.01.010.

[78] Wongtanakitcharoen, T. and A. E. Naaman. "Unrestrained early age shrinkage of concrete with polypropylene, pva, and carbon fibers." Materials and structures 40 (2007): 289-300. doi:10.1617/s11527-006-9106-z.

[79] Pelisser, F., A. B. D. S. Neto, H. L. L. Rovere and R. C. D. A. Pinto. "Effect of the addition of synthetic fibers to concrete thin slabs on plastic shrinkage cracking." Construction and Building Materials 24 (2010): 2171-76. doi:10.1016/j.conbuildmat.2010.04.041.

[80] Boghossian, E. and L. D. Wegner. "Use of flax fibres to reduce plastic shrinkage cracking in concrete." Cement and Concrete Composites 30 (2008): 929-37. doi:10.1016/j.cemconcomp.2008.09.003.

[81] Ruijie, M. A., J. Yang, Y. Liu and X. Zheng. "Influence of length-to-diameter ratio on shrinkage of basalt fiber concrete." Presented at 3rd International Conference on Applied Materials and Manufacturing Technology (ICAMMT, Changsha, PEOPLES R CHINA, 2017. IOP Publishing, 242, doi:10.1088/1757-899x/242/1/012027.

[82] Pacheco-Torgal, F., Y. Ding and S. Jalali. "Properties and durability of concrete containing polymeric wastes (tyre rubber and polyethylene terephthalate bottles): An overview." Construction and Building Materials 30 (2012): 714-24. doi:10.1016/j.conbuildmat.2011.11.047.

[83] Hawreen, A. and J. A. Bogas. "Creep, shrinkage and mechanical properties of concrete reinforced with different types of carbon nanotubes." Construction and Building Materials 198 (2019): 70-81. doi:10.1016/j.conbuildmat.2018.11.253.

[84] Dang, J., J. Zhao and Z. Du. "Effect of superabsorbent polymer on the properties of concrete." Polymers 9 (2017).

[85] Shen, D., X. Wang, D. Cheng, J. Zhang and G. Jiang. "Effect of internal curing with super absorbent polymers on autogenous shrinkage of concrete at early age." Construction and Building Materials 106 (2016): 512-22. doi:10.1016/j.conbuildmat.2015.12.115.

[86] Wyzykowski, M., S. I. Igarashi, P. Lura and V. Mechtcherine. "Recommendation of rilem tc 260-rsc: Using superabsorbent polymers (sap) to mitigate autogenous shrinkage." Materials and Structures/Materiaux et Constructions 51 (2018). doi:10.1617/s11527-018-1241-9.

[87] Wang, F., Y. Zhou, B. Peng, Z. Liu and S. Hu. "Autogenous shrinkage of concrete with super-absorbent polymer." ACI Materials Journal 106 (2009): 123. doi:10.14359/56458.

[88] Won, J. P., J. H. Kim, C. G. Park, J. W. Kang and H. Y. Kim. "Shrinkage cracking of styrene butadiene polymeric emulsion-modified concrete using rapid-hardening cement." Journal of Applied Polymer Science 112 (2009): 2229-34. doi:10.1002/app.29733.

[89] Habib, A., U. Yildirim and O. Eren. "Mechanical and dynamic properties of high strength concrete with well graded coarse and fine tire rubber." Construction and Building Materials, 246 (2020): 118502. doi:10.1016/j.conbuildmat.2020.118502.
[90] Habib, A., U. Yildirim and O. Eren. "Column repair and strengthening using rc jacketing: A brief state-of-the-art review." Innovative Infrastructure Solutions 5 (2020): 1-11. doi:10.1007/s41062-020-00329-4.

[91] Habib, A., U. Yildirim and O. Eren. "Properties of high-strength concrete containing well graded rubber particles." IOP Conference Series Materials Science and Engineering 800 (2020): 012018. doi:10.1088/1757-899x/800/1/012018.

[92] Sukontasukkul, P. and K. Tiamlom. "Expansion under water and drying shrinkage of rubberized concrete mixed with crumb rubber with different size." Construction and Building Materials 12 (2008): 520-26. doi:10.1016/j.conbuildmat.2011.07.032.

[93] Si, R., S. Guo and Q. Dai. "Durability performance of rubberized mortar and concrete with naoh-solution treated rubber particles." Construction and Building Materials 153 (2017): 496-505. doi:10.1016/j.conbuildmat.2017.07.085.

[94] Najim, K. B. and M. R. Hall. A review of the fresh/hardened properties and applications for plain-(prc) and self-compacting rubberised concrete (scrc). 24. Elsevier Ltd, 2010, 2043-51. doi:10.1016/j.conbuildmat.2010.04.056.

[95] Saito, M., M. Kawamura and S. Arakawa. "Role of aggregate in the shrinkage of ordinary portland and expansive cement concrete." Cement and Concrete Composites 13 (1982): 115-21. doi:10.1016/0958-9465(91)90006-4.

[96] Malkawi, A. B., M. Habib, J. Aladwan and Y. Alzubi. "Engineering properties of fibre reinforced lightweight geopolymer concrete using palm oil biowastes." Australian Journal of Civil Engineering 18 (2020): 82-92. doi:10.1080/14488353.2020.1721954.

[97] Nuruddin, M. F., A. B. Malkawi, A. Fauzi, B. S. Mohammed and H. M. Almattarneh. "Geopolymer concrete for structural use: Recent findings and limitations." Presented at IOP Conference Series: Materials Science and Engineering, 2016. 133, doi:10.1088/1757-899x/133/1/012021.

[98] Malkawi, A. B., M. Habib, Y. Alzubi and J. Aladwan. "Engineering properties of lightweight geopolymer concrete using palm oil cinder aggregate." International Journal 18 (2020): 132-39. doi:10.21660/2020.65.89948.

[99] Fauzi, A., M. F. Nuruddin, A. B. Malkawi, M. M. A. B. Abdullah and B. S. Mohammed. Effect of alkaline solution to fly ash ratio on geopolymer mortar properties. 2017, doi:10.4028/www.scientific.net/kem.733.85.

[100] Nuruddin, M. F., A. B. Malkawi, A. Fauzi, B. S. Mohammed and H. M. Al-Mattarneh. "Effects of alkaline solution on the microstructure of hca geopolymers." Presented at Engineering Challenges for Sustainable Future - Proceedings of the 3rd International Conference on Civil, offshore and Environmental Engineering, ICCOEE 2016, 2016. doi:10.1201/b21942-102.

[101] Wallah, S. E. "Drying shrinkage of heat-cured fly ash-based geopolymer concrete." Modern Applied Science 3 (2009). doi:10.5539/mas.v3n12p14.

[102] Nuruddin, M. F., A. B. Malkawi, A. Fauzi, B. S. Mohammed and H. M. Al-Mattarneh. "Evolution of geopolymer binders: A review." Presented at IOP Conference Series: Materials Science and Engineering, 2016. 133, doi:10.1088/1757-899x/133/1/012052.

[103] Collins, F. and J. G. Sanjayan. "Effect of pore size distribution on drying shrinking of alkali-activated slag concrete." Cement and Concrete Research 30 (2000): 1401-06. doi:10.1016/s0008-8846(00)00327-6.

[104] Mermerdaş, K., Z. Algun and Ş. Ekmen. "Experimental assessment and optimization of mix parameters of fly ash-based lightweight geopolymer mortar with respect to shrinkage and strength." Journal of Building Engineering 31 (2020): 101351. doi:10.1016/j.jobe.2020.101351.