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Peruvian Subduction Surface Model for Seismic Hazard Assessments

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Received 30 March 2019; Accepted 12 May 2019

Abstract

Throughout the years seismic hazard calculations in Peru have been developed using area sources models, having to date a great variety of models, however, since they are discretized planar models, they cannot adequately represent the continuity and subduction characteristics of the Nazca Plate. The main objective of this work is the developing of a surface subduction model (SSM), useful for seismic hazard assessments as well as the revision and control of previous models used in this sort of assessments. In this study a spatial interpolation was performed employing the Local Polynomial Interpolation method to capture short-range variation in addition to long-range trends. The data base is based on the compilation of seismic catalogs from Peruvian and international institutions such as the IGP, the USGS, the ISC and others, subsequently, in order to have independent events the elimination of duplicate events, aftershocks and foreshocks was carried out. Then, by interpolation of the focal depths of the independent events, a subduction surface model (SSM) was generated as well as a Standard Error Surface which supports a good correlation of the model. Furthermore, 14 transversal sections of the SSM was employed to compare with the hypocenter’s distributions, evidencing a good correlation with the spatial distribution of the events, in addition to adequately capturing the subduction characteristics of the Nazca Plate. Finally, a comparison was made between 2 Peruvian area models for seismic hazard and SSM developed in the present research, evidencing that seismic source models of the area type have deficiencies mainly in the depths they consider, thus is recommended the use of the present model for future seismic hazard assessments.

Keywords: Subduction Surface Model; Seismic Hazard; Seismic Sources.

1. Introduction

Definition of seismogenic source are one of the most crucial parts in probabilistic seismic hazard assessments because it models the spatial distribution of earthquake events. Castillo and Alva [1] proposed the first seismotectonic model for seismic hazard assessments in Peru, his models was composed by 20 seismogenic area sources of which 12 were to model the subduction and the rest a continental sources. As a part of their work, the author developed the first national seismic hazard assessment in Peru. This model was extended employed, many years later, Bolanos and Monroy [2] developed an actualization of the national seismic hazard using this model.

Later, Gamarra [3] developed an actualized a model of 20 area sources, 14 of these were subduction sources. Some years later, Aguilar et al. [4] presented an actualization of the models trying to capture the spatial distribution of earthquakes that previous models could not adequately represent. This last model considers 29 area sources and 20 of them are used to model the intraplate and interface seismic zones. Although each time the models presented a greater discretization of the area, the sources in order to better represent the subduction characteristics, these do not necessarily

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doi: http://dx.doi.org/10.28991/cej-2019-03091305

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fit into the characteristics of the Nazca Plate. Consequently, it is necessary to develop a model that can reveal the depth and distribution of the hypocenters of seismic events throughout Peru and that can be used with new trends in seismic hazard calculations, such as smoothed seismicity models.

The present work proposes the development of a subduction surface that can capture the neotectonics characteristics of Nazca Plate subduction, based on the interpolation of the hypocenters of a catalog that includes independent seismic events. This catalog was obtained after an elimination of foreshocks, aftershocks and duplicate events of a compilation of catalogs of several entities, such as the International Seismological Center (ISC), the Geophysical Institute of Peru (IGP), the United States Geological Survey, among others. Subsequently, a comparison of the resulting surface with the 2 seismotectonic models most used in the seismic hazard studies in Peru is made. Results show that the subduction model here developed, fits much better subduction mechanism than the previous ones.

2. Theatrical Background

2.1. Earthquake’s Terminology

There is a range of terminologies used to describe the location of an earthquake, since it is the result of the rock breaking along a fault, which may involve several square kilometers of the fault plane surfaces. The point where the rupture begins is the hypocenter or the focus and the projection on the point on the surface is like the epicenter. The distance from the epicenter to the focus is known as hypocentral depth. The distance between the epicenter and an observer is the epicentral distance, besides, the hypocentral or focal distance is the corresponding distance between the focus and the observer [5], see Figure 1.

![Figure 1. Spatial notation of earthquakes](image)

2.2. Spatial Distribution of Earthquakes in Peru

The seismicity associated with subduction events is concentrated throughout the interaction between the Nazca Plate and the South American Plate. The border between the Nazca Plate and the South American Plate in this region is demarcated by the Peru-Chile trench, which is 90 km west of the Peruvian coast. The seismic events from interface subduction are found in the western edge of Peru, mainly between the Peru-Chile trench and the coastal littoral, while the seismicity with focus at intermediate depth (60 km < h ≤ 350 km) is irregularly distributed beneath the continent.

On the other hand, deep-focus earthquakes (h ≥ 350 km) occur at the edges of the submerged sheet that is in contact with the mantle material at great depths (hundreds of km); there this edge crumbles inside the mantle releasing seismic energy in the form of waves [6].

2.3. Tectonic Structures of the Nazca Plate

Several tectonic structures are found off the Peruvian coast, some of which influence the spatial distribution of earthquakes, such as the Carnegie Ridge and the Nazca Ridge. In Figure 2 the tectonic features of the Nazca Plate are shown.

The Carnegie Seismic Ridge is a high bathymetric Nazca Plate originated in the area of the Galapagos Islands, has an approximate direction EW, and enters the zone of subduction between 1°N and 2°S of latitude. The Ecuadorian continental margin rises along the Carnegie collision area with the trench [7].
The Grijalva escarpment, an old fracture zone N60°E in the Farallon Plate, is interpreted as the southern half of the fault trace from which the Nazca Plate was detached. Additionally, exist an anomaly beneath northern Peru, called the Inca Plateau and it could be the responsible of a low seismicity activity in the northeastern of Peru [8, 9].

The Mendana fracture is one of the tectonic features of the Nazca Plate that is in the extreme west of the central region of Peru between latitudes 11° S and 15° S. It has a NE-SW orientation, that is, perpendicular to the line of the Peruvian-Chilean trench; with an approximate length of 1100 km, an average height of 1000 m above the oceanic crust and a width of approximately 80 km. Its origin is associated with an old zone of plate divergence [10].

The Nazca Ridge, is an old oceanic mountain range that is observed in the bottom of the sea, collides with the South American Plate and is located in the extreme NO of the South region of Peru in front of the department of Ica with great influence in the tectonic constitution of the western part. It has a NE-SW orientation perpendicular to the line of the Peruvian-Chilean trench between the latitudes 14° S and 16° S in such a way that its NE end is located in front of the department of Ica where it has a width of approximately 220 km above the 1.5 km elevation, however, its width and altitude gradually decrease towards its SW end [11].

At 450 km from the Nazca Ridge, the Nazca fracture is in front of the department of Arequipa, being the most remarkable bathymetric characteristic. The fracture zone has a depressed valley structure (~ 1.2 km depth) along the fault line [12], which is aligned in a NE-SW direction, perpendicular to the trench. Robinson et al. [13] link the fracture zone as a rupture barrier of the Arequipa earthquake in 2001.

3. Develop of a Subduction Surface Model

3.1. Hypocenters Data

In the present work, the information available from national and international catalogs such as the Geophysical Institute of Peru, International Seismological Center (ISC) (ISC), the United States Geological Survey (USGS), has been compiled. in English), the National Earthquake Information Center (NEIC), the National Oceanic and Atmospheric Administration (NOAA), the Global Centroid Moment Tensor of Harvard University (GCMT), among others, between
the meridians 66 ° W and 84 ° W Greenwich and the parallels 4 ° N and 23 ° S in order to obtain the most complete seismic information from a magnitude Mw ≥ 4.0.

The methodologies of elimination of foreshocks and aftershocks events used in this work correspond to the Reasenberg’s second-order moment analysis [14] and the spatial, temporal and magnitude criterion developed by Maeda [15]. Furthermore, since it is a collection of events, there are cases in which the same event is registered by 2 or more different entities, for this reason an elimination of duplicate events was also carried out. The final seismic catalog consists of 11688 earthquakes with a magnitude Mw ≥ 4.0, Figure 3 shows the catalog with seismic events of magnitude greater than 5 Mw, where the size and chromatic scales represent the magnitude and depth range respectively.

![Seismic catalog of events 1555-2017 considering a magnitude Mw≥5](image)

**3.2. Spatial Interpolation**

The interpolation tools make use of spatial and temporally distributed data about the phenomenon under study in order to make a prediction of the areas where no measurements have been recorded. Specifically, for the case of seismic events are distributed spatially by location of the epicenter and hypocentral depth. There are multiple methods for obtaining a prediction for each location. Local methods assume that each point influences the resulting surface only up to a certain finite distance. Values at different unsampled points are calculated by functions with different parameters, and the continuity condition between these functions is defined only for some approaches [16].

The local polynomial interpolation method performs a polynomial fit to the specified order using points only within the defined neighborhood. The neighborhoods overlap, and the value used for each prediction is the value of the adjusted polynomial in the center of the neighborhood. Figure 4 shows a surface that has a variable shape, a landscape that tilts, levels and slopes again, where multiple polynomial planes represent the surface with better precision compared to a global interpolation.
To obtain the subduction surface, the local polynomial interpolation algorithm of ArcGIS was chosen using the spatial distribution, latitude, longitude and focal depth corresponding to each event. The other parameters were chosen based on the recommendations of the software developers [16] and an iterative process, which are presented in the Table 1.

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<th>Value/Description</th>
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### 3.3. Surface Subduction Model

The elevation views of the obtained subduction surface model (SSM) is shown in Figure 5, while in Figure 6 a plan view with isolines spaced each 1km of depth, can be appreciated. At first sight, the subduction model allows to differentiate different zones whose subduction has peculiar characteristics. While in the north and center of Peru the subduction has a low slope, in the south it becomes more abrupt, generating a marked transition zone between the limits of the central and southern zones, a more detailed description will be made in the following sections.
In order to know the precision of the prediction, a prediction standard error surface was created (Figure 7), which indicate the uncertainty associated with the value predicted for each location. As it could be noticed, the standard error of the SSM is lower than 10 km which means a strong correlation of the model with the spatial distribution of the hypocenters, except when the prediction reaches high depths in zones where information of the events is lower.

Figure 6. Plan view of the obtained subduction surface

Figure 7. Standard error of the subduction surface model.
Additionally, in order to verify the quality of the adjustment, 14 sections with a width of 1º have been made, which in turn are grouped into 4 zones which are shown in Figure 8, the subduction region near Ecuador (red lines), the northern zone (purple lines), the central zone (green lines) and the southern zone of Peru (blue lines). The spatial distribution of hypocenters and the Surface subduction Model (SSM) shows a strong correlation and let appreciate the different ways in which Nazca Plate subducts the continental crust.

![Figure 8. Selected Cross Sections](image)

3.4. Northern Zone of Ecuador

In the northern zone of Ecuador represented by section 1 (See Figure 9), it can be seen that there is a continuous subduction up to 200 km depth with an angle between 25 and 30 º and a distance of 400 km from the trench, this behavior gradually varies towards the southern part of the equator becoming a subduction with an angle of 20 to 25 º as shown in sections 2 and 3. This behavior is consistent with what has been described by several authors [17-19], who determine a subduction angle in the range of 25 to 35 º between the parallels 1.5 º N and 2.5 º S, which subducts continuously up to around 200 km. In central part Ecuador, between 4ºS and 1ºS, the Nazca Plate undergoes a sharp contortion similar to that observed in southern Peru where at ~15ºS, this is because the presence of the ancient Farallon Plate [20].
3.5. Northern Zone of Peru

In the northern zone of Peru according to sections 4 through 6 of Figure 10, the subduction advances with an angle of ~20°, reaching an average depth of 150 km, depth at which it becomes a subduction horizontal sub to the 700 km from the trench.

3.6. Central Zone of Peru

The central zone of Peru maintains the subduction tendency of the northern zone, with the difference that the zone of initial subduction that advances with an angle of ~20°, reaches a depth of 100 km at distances between 200 and 250 km from the trench, from where it becomes a sub horizontal subduction until 500 and 600 km from the trench, where the subduction increases its slope, to then reach depths in the range of 600 and 700 km, where a deep seismicity is the
product of the contact with the Brazilian shield, allowing to extent the model until these depths, it can be appreciate in sections 7 to 10 of Figure 11.

Previous studies have shown that subduction of the Nazca Plate occurs at an approximate angle of 18º, reaching a depth of 100 km, from which it remains constant for about 300 km to subsequently resume its descent [12, 22], so the model developed is consistent with the subduction characteristics of the area.

Furthermore, it can be noticed that the model presents a practically flat subduction that moves away from the trench between the parallels 11º S and 16º S, which coincides with the entrance of the Nazca ridge to the coast and maintains a homogeneous depth between the 90 and 105 km deep. This behavior is consistent with other models, Phillips et al. [12] propose that the Nazca ridge reaches a constant depth of 100 km, while Antonijevic et al. [24] developed a model based on cutting wave velocities in addition to a conceptual model of dorsal formation, concluding that it subducts to a depth of 90 km.

3.7. Southern Zone of Peru

The marked difference between sections 10 and 11 denotes an abrupt change in the form of subduction is observed, coinciding with the intersection of the Nazca Ridge with the Peru Chile trench as was described in other studies [12, 21, 23, 25, 26]. Moreover, in this transition zone a decrease in seismicity is observed in the model, according to Dougherty and Clayton [25], this decrease in seismicity suggests a change in the structure of the plate between the normal subduction in the southeast and the sub horizontal in the northeastern zone. This variation in seismicity is likely related to the overthickened crust of the Nazca Ridge, compared to the normal oceanic crust on either side of the ridge [26].

The southern zone has a steeper subduction with an angle of 30 ° to 300 km depth which is consistent with that of previous studies [21, 22, 25]. As observed in sections 11 and 12 of Figure 12, earthquakes occur up to a distance of 450 km from the trench, while for sections 13 and 14 of the same figure, there is a large number of seismic records at distances from the trench of 550 km on average.
It should be mentioned that the locations of the earthquakes usually provide a first-order estimate of the geometry of the oceanic trench, however, currently there are more advances, but complicated methods that allow to study in a more adequate way the upper and lower limits of the subduction zone [12, 23, 25, 28], however, for seismic hazard assessments the SSM developed in this work can be used adequately.

4. Comparison of Seismotectonic Models

Seismogenic source are one of the most crucial parts in probabilistic seismic hazard assessments because it models the spatial distribution of earthquake events. Castillo and Alva [1] proposed the first seismotectonic model for seismic hazard assessments in Peru, his models was composed by 20 seismogenic area sources of which 12 were to model the subduction and the rest a continental sources. Gamarra [3] developed new seismotectonic model of 20 area sources, 14 of these were subduction sources which in contrast to Castillo and Alva [1] model, discretized better the subduction zone. Some years later, Aguilar et al. [4] present an actualized of the models trying to capture the spatial distribution of earthquakes that previous models could not adequately represent. This last model considers 29 area sources and 20 of them model subduction seismic zones, being the model that discretizes more than other the Nazca Plate subduction.

Figure 13 shows the comparison of the subduction surface generated with the sources of Gamarra [3] and Aguilar et al. [4]. As can be seen in the figure, both models show deviations from the SSM of the present investigation, however, Aguilar's sources adjust much better than those of Gamarra. Although the area sources proposed by Aguilar et al. fits better to the SSM in comparison to Gamarra’s source, for further works in seismic hazard assessments, the use of subduction surface models such as the one generated in the present investigation is recommended.
5. Conclusion

The subduction model developed in this research allows to estimate the distribution of earthquakes in areas where there is not enough data of earthquakes, being of great help both for the control of the geometries of seismogenic sources and for the generation of smoothed seismicity models, thus allowing the performance of seismic hazard assessments in a more reasonable way than simply use geometrical area sources.

The geometry of the resulting subduction surface represents adequately the tectonic features of the Nazca Plate, such as the Inca Plateau and Nazca ridge, both responsible for a markable flat subduction zone in the north and central zone in Peru, the last also divides the central and southern subduction zone of the Peruvian coast, the former having a sub-horizontal subduction and the second, a continuous subduction with an angle of 30° to reach considerable depths, all of these patterns we observe are consistent with the stated by several authors, as was detailed in the previous sections.

The seismicity of deep focus, product of the contact with the Brazilian shield, allows to suggest a geometry of subduction to great depths, nevertheless, for the purposes of estimation of the seismic assessments they do not have greater relevance.

6. Funding

This work was supported by the company ZER Geosystem and the Japanese Peruvian Center for Seismic Research and Disaster Mitigation (CISMID).

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Numerical Evaluation of Foundation of Digester Tank of Sewage Treatment Plant

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Received 21 January 2019; Accepted 17 April 2019

Abstract

In the present study the foundation of digester tank, main part of sewage treatment plant, is reanalyzed analytically and numerically to check the adequacy of such foundation to support superstructure loading. The foundation of digester tank consists of raft foundation and bored piles. The diameter of raft is 33 m and thickness of 1 m, while the piles are bored type of diameter 0.6 m and length 15 m. After testing eleven working piles, it is found that three piles cannot support a load of 1.5 times the working load (1305 kN) safely or in other words the factor of safety of these failed piles is less than 1.5. The results of piled pile tests are reanalyzed using two well-known methods, Davisson’s method and Brinch-Hansen method to check the ultimate carrying capacity of tested piles. Also, this paper includes analysis of previous soil investigation report and conducting additional soil investigation by drilling three boreholes to secure the soil parameters used in the analytical and numerical analysis of digester tank foundation. SAFE 12 software is used to analysis the foundation of structure as piled-raft instead of pile group to interest from the interaction between soil and raft foundation. The results of analysis showed that the piles failed in the tests can support its share of the superstructure load by a factor of safety 1.8 and the piles success in the field tests can support its share of the superstructure load by a factor of safety not less than 2.86. Also, the settlement under structure will be less than 100 mm, where using piled-raft analysis reduces the settlement to be within allowable limits.

Keywords: Pile; Raft; Foundation; Soil; Tank.

1. Introduction

The digester tank is considered important structures in comparison with other structures that are consist the sewage treatment plant. The digester tank is reinforced concrete tank and consists of three main parts. The upper part is of a conical shape with outer radius of 12.5 m and height of 6.5 m, the middle part has a cylindrical shape with outer radius of 12.5 m at its upper part (7.3 m height) and 12.8 m at its lower part (6.45 m height) and the lower part of the digester is of an inverted conical shape with outer radius of 12.8 m. The total height of the tank will be 29.25 m. The tank is supported by eight triangular radial walls that fixing the inverted conical base to the tank raft. The foundation of tank is a reinforced concrete raft of 16.5 m radius and 1 m thickness resting at a depth of about 2.5 m below the ground level. This raft is supported by 193 piles distributed in a radial direction. The bored piles of 600 mm diameter and 15 m length were casted in situ using reinforced concrete. The digester is currently under construction and 193 piles were constructed and the raft has been casted as well. The structural designer has defined the working load of each pile as 870 kN [1].

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http://dx.doi.org/10.28991/cej-2019-03091306

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Hirai [2] presented an analytical solution using Winkler model for the analysis of piled-raft foundation subjected to vertical loading in heterogeneous soils. The coefficient of vertical stiffness of the pile in is derived from Mindlin solution for the displacement of elastic continuum. The coefficient of vertical stiffness for the raft is expressed by the Muki solution for the 3-D elastic analysis. The relation between settlement and vertical load for the piled-raft system is obtained by using the recurrence equation of influence factors of the pile for each layer. The percentage of load carried by both piles and raft is represented by the vertical influence factors. The results calculated by the present method for piled-raft with nodular piles in heterogeneous soils well agreed with those obtained from field test and the finite element method.

Al-Kaisi et al. [3] studied the behavior of free-standing pile groups and piled-raft driven in clayey soil under axial loading. Different configurations of piles were tested such pile diameter, pile length, and spacing between piles. Also, they conducted tests on piles in cohesive soil of different shear strength. It is observed that piles exhibited a very high stiffness at initial loading stages till the settlement reaches 0.5 mm, but then the pile settled rapidly with small increment of the load. In addition, most of the load capacity of piles is mobilized at settlement of around (1–2) mm which is corresponding to 5 % of pile diameter. The undrained shear strength of clay has no significant effect on the mechanism of load transfer by piles. The load carrying capacity of pile group is equal to that of piled-raft foundation, where the interaction between piles and raft is not significant. Dezfooli [4] studied the effects of reinforcement elements such as geogrid in the cushion layer of the non-connected pile-raft foundation in sandy soil on the mechanism of load transfer and the shares of piles and raft from the total applied load. The effect of different parameters such as the spacing between piles, thickness of cushion, the number and length of geogrid layers on the load settlement of foundation system had been studied. The results showed that the lowest settlement observed in non-reinforced cases with an optimum cushion thickness and piles spacing, the lowest settlement is observed. Using the geogrid in the cushion layer causes increasing the bearing capacity and the share of the total load carried by the piles.

Many researchers conducted Mali and Singh [5] 3D numerical analysis to understand the settlement, load-sharing, bending moment and shear force behavior of piled-rafts founded on different soil profiles and different loading configurations, and different piled-raft configurations. The results of these studies showed that as the pile spacing increases, the average settlement decreases significantly for different soil profiles soil profile and it is noted to be lesser for uniform piled-raft configurations. Also, the load-sharing ratio increases with increases the pile spacing. The maximum bending moment and shear force are noted to be less in piled-raft foundation than that of raft foundation or pile group analysis [5-9]. The present study is a case study focused on reanalyzed a constructed raft and piles to calculate the allowable carrying capacity of piles and apply SAFE 12 software to analyze the system as piled-raft instead of analysis the system as pile group. This problem raised after testing eleven working piles in the field, it is found that three piles cannot support a load of 1.5 times the design load of 1305 kN safely or in other words the factor of safety of these failed piles is less than 1.5 [10-11]. Therefore, the problem reanalyzed to check if the casted bored piles and raft foundation can support the superstructure load or not. In case the pile group failed to withstand the applied loads will be removed otherwise if it can support the loads with an allowable factor of safety, the construction process will be continued. This project can be considered is trial to analyze the foundation as piled-raft instead of pile group which is considered of of the sustainability development aspects by changing the design criteria.

2. Site Description and Methodology of Analysis

According to the site investigating report and soil tests that performed on samples obtained from 12 boreholes drilled to a depth of 20 m each, the soil stratification can be described as follows [12]:

a) The surface layer starting from the natural ground surface consists of brownish to grayish silty clay soil (with sand CL-CH), soft to medium consistency, this layer extended to a depth of (0.0-4.5) m.

b) A layer of greenish silty sand soil (SW-SP) with clay, loose to medium dense, this layer extended to a depth of (2.5-7.5) m.

c) A layer consists of brownish to grayish silty clay soil with sand (CL-CH), soft to medium to stiff in consistency; this layer extends up to depth of (0.0-15.5) m.

d) A layer of greenish silty sand soil with content gypsum (SW-SP), dense to very dense, this layer extended to the end of boring (14.0-20.0) m.

e) A layer consists of greenish silty clay soil with sand (CL), stiff consistency, this layer extended to the end of boring (18.5-20.0) m.

This consequent changes or sub-soil strata is related to way of sedimentation. The soil investigation suggested using the following geotechnical data:

a) The allowable bearing capacity of shallow foundation at depth ranging from 2 to 3 m below the ground level is ranging from 80 to 98 kPa.
b) The allowable carrying capacity of bored piles of 600 mm diameter and 15 m length is 1092 kN which based on a factor of safety of 2.5, so that the ultimate carrying capacity of such piles is 2730 kN.

The methodology used in the present work can be illustrated in the flowchart shown in Figure 1.

![Flowchart of methodology used in the analysis of case study](image)

Figure 1. Flowchart of methodology used in the analysis of case study

It is believed that the suggested carrying capacity is overestimated as none of the pile tests indicates such high values for the ultimate carrying capacity. For this reason, it is suggested to perform an additional soil exploration by drilling another three exploring boreholes which may help in obtaining more precise soil parameters to be used in the analysis of the problem in hand. Two of boreholes were drilled to a depth of 20 m and the third borehole was drilled to a depth of 10 m [13]. The data obtained from the additional soil investigation suggested that the allowable bearing capacity of shallow foundation at depth 1.5 m is ranging from 55-80 kPa and at depth 4.5 m is ranging from 57.5-110 kPa. Also, the bored pile of 600 mm diameter and 15 m depth has an allowable carrying capacity of 710 kN. This conservative value of allowable carrying capacity is almost consistent with the pile tests results.

3. Geotechnical Parameters of Soil

In order to perform a sound analysis to the problem in hand, it is very important to define reasonable values for the soil parameters that will be used in the analysis. After careful reviewing of the site investigating reports, the parameters given in Table 1 are considered as representative and conservative values for the soil parameters.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>The undrained shear strength (Su)</td>
<td>34 kPa</td>
</tr>
<tr>
<td>The compression index (Cc)</td>
<td>0.10534</td>
</tr>
<tr>
<td>The swelling index (Cs)</td>
<td>0.01115</td>
</tr>
<tr>
<td>The initial void ratio (e₀)</td>
<td>0.735</td>
</tr>
<tr>
<td>The preconsolidation pressure (Pc)</td>
<td>111 kPa</td>
</tr>
<tr>
<td>The constrain modulus (Eₘₙₖ)</td>
<td>8050 kPa</td>
</tr>
<tr>
<td>The dry unit weight (γₐ)</td>
<td>15.26 kN/m³</td>
</tr>
<tr>
<td>The saturated unit weight (γₛₙₐₖ)</td>
<td>19.76 kN/m³</td>
</tr>
<tr>
<td>The Poisson’s ratio (µ)</td>
<td>0.4 (assumed)</td>
</tr>
<tr>
<td>The water table level</td>
<td>1.5 m below EGL</td>
</tr>
</tbody>
</table>
The calculated soil parameters that will be considered in the analysis are [14]:

- The modulus of elasticity of soil (E) calculated as follows to be is 3760 kPa:
  \[ E = E_{oed} (1 + \mu)/(1 - 2\mu) \]  
  \( (1) \)

- The shear modulus (G) can be obtained by using three different correlation expressions [14, 15]:
  \[ G = E/2(1 + \mu) \]  
  \( (2) \)
  \[ G = (100 - 300)Su \]  
  \( (3) \)
  \[ G = 0.6 \times 25^{Su/100} \, (G \text{ in MPa}) \]  
  \( (4) \)

The first equation gives a value of G about 1343 kPa but it depends on the elastic properties of soil and doesn’t account for the undrained shear strength (Su). The second equation and by using its lower limit gives a value of 3600 kPa for G and the third equation results in a value of 1850 kPa for G. By considering the three values of G, a conservative value of G = 2000 kPa is quite reasonable to be considered as a representative value for the soil layer in the analysis.

- Regarding the modulus of subgrade reaction of the supporting soil (Ks) in the site, the geotechnical report assumes that at depth of 2.5 m, Ks is ranging from 2000 to 13000 kN/m² [13]. This value is obtained by using the following formula:
  \[ K_s = 40 \times F_5 \times q_{all} \]  
  \( (5) \)

There is another equation to calculate Ks:

\[ K_s = E/B(1 - \mu^2) \]  
\( (6) \)

Equation 6 gives a much-underestimated value of Ks = 136 kN/m³. For silty clay soils, the value of Ks is ranging from 2000 to 20000 kN/m³, therefore the lower limit value of Ks = 2000 kN/m³ will be used in the analysis.

4. Theoretical Analysis of Piled-Raft Foundation

Before starting the theoretical analysis, it is important to calculate the applied load on the soil by the primary digester tank. The main load of this structure is self-weight plus the weight of water inside. The live load which implicitly considered constitutes a very low portion of the total weight of the structure. The total weight of the concrete tank and the water at its maximum capacity is about 160000 kN and the applied stress on the soil will be 187 kPa by considering the raft area of 855 m². Regarding the wind load, a calculation was performed by considering a wind speed of about 85 mph and an importance factor of 1.15 [16]. It is found that for such structure of height about 30 m, the wind pressure is ranging from 1.4 kPa at its top to about 1 kPa at its bottom. This will give a resultant horizontal force of about 1000 kN and a resulting moment on the foundation of about 20000 kN.m. To calculate the eccentricity (e) of the total applied loads, the value of moment is divided by the axial force (16000 kN) to get the eccentricity value (e = 0.125 m) which is less than 1/6 of the footing width. Therefore, the wind load can be neglected in the analysis. In the beginning, the foundation of the structure will be analyzed as raft foundation at depth 2.5 m. The ultimate bearing capacity (q_{ult}) of such raft can be calculated as follows [17]:

\[ q_{ult} = 5 \times Su \times \left(1 + \frac{0.2B}{L}\right) \left(1 + \frac{0.2D}{L}\right) + \gamma D \]  
\( (7) \)

In which B and L are width and length of the raft and D is depth of foundation placement. For circular footing, both L and B are substituted by the raft diameter. Equation 7 gives a value of 220 kPa, where the foundation can carry applied stress with a factor of safety of 1.18. Due to its large diameter, raft foundation usually causes a settlement more than that permissible by different codes of practice. Therefore, the factor of safety should be increased to control the foundation settlement to be within allowable limits. The tolerable settlements, total and differentiable, of different types of foundation constructed in different types of soils based on the experience of many agencies and persons are given in Table 2.

<table>
<thead>
<tr>
<th>Type of footing</th>
<th>Type of soil</th>
<th>Total settlement, mm</th>
<th>Differentiable settlement, mm</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isolated and Strip</td>
<td>Sand</td>
<td>25</td>
<td>-</td>
<td>[18]</td>
</tr>
<tr>
<td>Slab and raft</td>
<td>Sand</td>
<td>50</td>
<td>-</td>
<td>[19]</td>
</tr>
<tr>
<td>Isolated and Strip</td>
<td>Sand</td>
<td>40</td>
<td>51</td>
<td>[20]</td>
</tr>
<tr>
<td>Slab and raft</td>
<td>Sand</td>
<td>45-65</td>
<td>51-76</td>
<td>[20]</td>
</tr>
<tr>
<td>Isolated and Strip</td>
<td>Clay</td>
<td>65</td>
<td>76</td>
<td>[20]</td>
</tr>
<tr>
<td>Slab and raft</td>
<td>Clay</td>
<td>65-100</td>
<td>76-126</td>
<td>[20]</td>
</tr>
</tbody>
</table>
It’s recommended to adopt the values presented by Skempton and McDonald [20] in checking the settlement of the foundation. Regarding the circular raft of the primary digester tank, the anticipated settlement can be calculated as follows [21]:

$$ Sc = \frac{Cc}{1 + e_o} \cdot H \cdot \log \left( \frac{\sigma_o + \Delta \sigma_v}{\sigma_o} \right) \quad (8) $$

Considering normally consolidated clay of thickness, $H = 17.75 \, \text{m}$, $\sigma_o = 104 \, \text{kPa}$, and $\Delta \sigma_v = 116 \, \text{kPa}$ (structure load transferred to the mid of clayey layer by using method of 2V:1H). The resulting settlement is 350 mm which much greater than that allowed (65-100 mm). In such situation, piles are usually used and the foundation should be considered as piled-raft. The main function of using piles in such foundation is to reduce the settlement of the raft to its allowable limit and to increase the factor of safety of soil bearing capacity. It is documented that when the raft diameter (or width) is greater than the pile length the pile will no longer work as pile group, rather the foundation will work as piled-raft. To analyze the piled-raft foundation, it is important to calculate the pile stiffness and the raft stiffness individually then calculate the piled-raft stiffness. Regarding the piles (diameter = 600 mm and length = 15 m) installed in such soil, the following equation is used to define whether the pile is long (flexible) or short (rigid) pile. For short rigid piles [22-24]:

$$ \frac{L}{D} < 0.25 \frac{\sqrt{E_p}}{G} \quad (9) $$

$L/D$ is 25 which is less than the right-hand side term (27), therefore the pile is considered as short (rigid) pile and the stiffness of an individual pile ($K_{p1}$) is calculated by the Equation 10 [25]:

$$ K_{p1} = \frac{2D}{1 - \mu} G + \frac{2\pi GL}{\zeta} \quad (10) $$

Where ($\zeta$) value is ranging from 3-5 and considered as 4 in this analysis. The calculated value of ($K_{p1}$) is 51124 kN/m. The stiffness of 193 piles ($K_p$) cannot be considered as the sum of individual pile stiffness ($K_{p1}$) because of the interaction between piles. The suggested equation to calculate the stiffness of the whole group of piles ($K_p$) is [19]:

$$ K_p = K_{p1} \left( \text{No of Piles} \right)^\beta \quad (11) $$

A reasonable value of $\beta = 0.66$ can be considered in the analysis. This yields a value of $K_p$ of about 1648500 kN/m [26].

The raft stiffness ($K_r$) is calculated as follows [26]:

$$ K_r = 2 \frac{DG}{(1 - \mu)} \quad (12) $$

Equation 12 gives $K_r = 220000 \, \text{kN/m}$. The piled raft stiffness ($K_{pr}$) is calculated by using:

$$ K_{pr} = \frac{K_p + (1 - 2 \alpha_{pr})K_r}{1 - \alpha_{pr}^2 \left( \frac{K_r}{K_p} \right)} \quad (13) $$

Where ($\alpha_{pr}$) represents the factor of interaction between piles and raft, its value can be considered as 0.8. The calculated value of $K_{pr}$ is about 1659190 kN/m. Considering an applied load on the piled raft of 160000 kN, the resulting displacement will be 96 mm which is much less than that of raft alone and within the acceptable limits.

5. Analysis of Pile Test Results

As mentioned earlier, the circular raft is supported by 193 piles of 600 mm diameter and 15 m length. Eleven working piles were tested by considering the working load of each pile is 870 kN. The maximum axial loading reached in the pile tests is 1305 kN that represents 1.5 times the working load. By dividing the total applied load of tank (160000 kN) on the total number of piles, the share of each pile from load is about 830 kN. Accordingly, it is thought that the structural designer presumed that the piles will work as (pile group) with a group efficiency of 95%. The pile test reports indicate that three of the piles cannot carry a working load of 870 kN with a factor of safety (SF = 1.5) and mentioned that these piles are considered as failed piles. The report of pile tests did not refer to the adopted criteria to define neither the pile working load nor its ultimate load. According to the adopted soil parameters, the ultimate carrying capacity of pile can be calculated according to Equation 14 as follows [27, 28]:

$$ P_{ult} = \pi D L S_u + \frac{\pi D^2}{4} N_c S_u \quad (14) $$

Where

$D$ is the diameter of pile;
L is the length of pile;
Su is the undrained shear strength of soil;
Nc is the bearing capacity constant.

Considering Nc = 9, the ultimate carrying capacity of pile (Pult) will be about 1047 kN. If the factor of safety is assumed to 2, the allowable carrying capacity will be about 524 kN. Therefore, the assumed working load of each pile (870 kN) could be an overestimated value. To re-analyze the pile test results, two criteria have been adopted to calculate the ultimate carrying capacity of the pile from working pile test results. The used criteria are Davisson’s criteria and Brinch-Hansen (1963) criteria. The results of six piles tests are re-analyzed; two of the failed piles, namely Pile No. 138 and Pile No. 184 and four of the piles that passed in working piles tests, Pile No. 20, Pile No. 22, Pile No. 70, and Pile No. 142.

The calculated values of ultimate carrying capacity by Davisson’s criteria and Brinch-Hansen criteria of six of the tested piles are given in Table 3. It can be noticed that Davisson’s method gives a conservative value for the ultimate carrying capacity and cannot predict that value when it exceeds the maximum load that reached during the pile test. Since the number of tested piles is limited (only eleven piles), it is important to generalize a representative value for all the piles that passed the test and another value for all the failed piles. To be more reasonable, an averaging for the values obtained by the two adopted methods, Davisson and Brinch-Hansen, will be made then another averaging for the tested piles. This will result in an average value for the ultimate carrying capacity for the piles that passed the test of about 1380 kN and that value for the (failed) piles is 815 kN.

### Table 3. The values of ultimate carrying capacity as obtained from pile test results

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Davisson's Method Pult (kN)</th>
<th>Brinch-Hansen Method Pult (kN)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>P20</td>
<td>&gt;1305</td>
<td>1643</td>
<td>Passed</td>
</tr>
<tr>
<td>P22</td>
<td>1152</td>
<td>1360</td>
<td>Passed</td>
</tr>
<tr>
<td>P70</td>
<td>1222</td>
<td>1410</td>
<td>Passed</td>
</tr>
<tr>
<td>P138</td>
<td>576</td>
<td>872</td>
<td>Failed</td>
</tr>
<tr>
<td>P142</td>
<td>1300</td>
<td>1580</td>
<td>Passed</td>
</tr>
<tr>
<td>P184</td>
<td>726</td>
<td>1080</td>
<td>Failed</td>
</tr>
</tbody>
</table>

6. Numerical Analysis of Tank Foundation

In this analysis, the computer program SAFE 12 [26] is used to model and analyze the foundation of tank. Only the piled-raft foundation will be modelled as reinforced concrete material with a unit weight of 24 kN/m³ and the tank and eight triangular walls supporting the tank will be represented as vertical loads applied on the circular raft. This simulation disregards the additional stiffness resulting from the structure of tank and could be in the safe side as the adopted stiffness of the raft is less than its actual value. In the program SAFE 12, each pile is modelled as an individual spring of a certain stiffness value $K_p$ and does not taken in the consideration the interaction effect between the piles. Therefore, it is important to define an equivalent value of $K_p$ that consider the interaction between the piles as a whole, pile group. In this analysis the pile test results will be adopted to define this value of pile stiffness because it is more reasonable than that obtained by theoretical analysis. After reviewing the pile test results, it is found that the secant stiffness values are ranging from 40 kN/mm for the failed piles and more than 240 kN/mm for the piles passed in tests. Therefore, an average value of 140 kN/mm will be considered as a conservative value as the number of failed piles is less than half of the total tested piles. To account for piles interaction, the equivalent value of pile stiffness that will be input in the analysis is only 10 kN/mm which is almost equal to $K_p = (193)^{0.5}/193$ [27, 28]. The value of soil subgrade reaction is also required by the program SAFE 12 and the adopted value is 2000 kN/m².

Figure 1 shows the contours of raft displacements resulted from the weight of tank. It can be noticed that the displacement is almost uniform with an average value of 43.8 mm. The maximum value is 44.2 mm occurs not at the raft center, as expected, but at a radial distance of about 7 m from the center which is the edge of triangular wall supporting the tank. This is mainly because of the smaller pile spacing close to raft center. The minimum displacement value occurs at the raft edge as expected with a value of 43.5 mm. The average value of displacement is within the acceptable limits (less than 50 mm) as mentioned earlier in Table 2, more than 90% of this displacement will take place during the period of construction which resulted from the digester tank self-weight and equipment plus the weight of water inside the tank. The differential settlement is about 0.7 mm which is very small and expected to be reduced if the stiffness of the tank is added to the raft stiffness. The contour map of resulting soil pressure is shown in Figure 2. It can be noticed that the distribution of subgrade soil pressure is consistent with the raft displacement. The average value of soil pressure is about 87.6 kPa which is less than the allowable bearing capacity value obtained by the theoretical analysis.
Figures 3 and 4 show the value of applied load on each of the 193 piles that supported the raft foundation. Table 4 contains the values of the axial load carried by each pile. It can be noticed that the value of this load varies from 440 kN to about 483 kN depending on the location of pile. Considering the average ultimate load values for both failed and passed piles, the average value of factor of safety (SF) for the failed piles is about 1.85 and that for the piles that passed test is 2.86. These values of factor of safety are based on all piles have the same stiffness as mentioned earlier, but it is well known that the load carried by each pile is proportional to its stiffness. Therefore, stiffer piles will carry more load and then the factor of safety will accordingly decrease. In the contrast to that, for piles of less stiffness, the carried load will decrease then the factor of safety will increase, which corresponding to the behavior of the failed piles which have a less stiffness value. It is expected therefor that both failed and passed piles will have a reasonable value of factor of safety of 2 or more.
Figure 3. The pile loading (right sector of the raft)

Figure 4. The pile loading (lower sector of the raft)

Table 4. The load carried by each pile obtained from analysis by SAFE 12 software

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Pile Load kN</th>
<th>Pile No.</th>
<th>Pile Load kN</th>
<th>Pile No.</th>
<th>Pile Load kN</th>
<th>Pile No.</th>
<th>Pile Load kN</th>
<th>Pile No.</th>
<th>Pile Load kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>482.709</td>
<td>40</td>
<td>443.131</td>
<td>79</td>
<td>442.098</td>
<td>118</td>
<td>470.633</td>
<td>157</td>
<td>458.692</td>
</tr>
<tr>
<td>2</td>
<td>455.56</td>
<td>41</td>
<td>446.283</td>
<td>80</td>
<td>455.625</td>
<td>119</td>
<td>441.6</td>
<td>158</td>
<td>445.586</td>
</tr>
<tr>
<td>3</td>
<td>452.216</td>
<td>42</td>
<td>445.498</td>
<td>81</td>
<td>443.907</td>
<td>120</td>
<td>452.201</td>
<td>159</td>
<td>451.16</td>
</tr>
<tr>
<td>4</td>
<td>455.56</td>
<td>43</td>
<td>450.905</td>
<td>82</td>
<td>440.996</td>
<td>121</td>
<td>442.631</td>
<td>160</td>
<td>445.575</td>
</tr>
<tr>
<td>5</td>
<td>452.216</td>
<td>44</td>
<td>445.499</td>
<td>83</td>
<td>443.908</td>
<td>122</td>
<td>438.878</td>
<td>161</td>
<td>458.687</td>
</tr>
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<td>455.56</td>
<td>45</td>
<td>446.283</td>
<td>84</td>
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<td>123</td>
<td>440.177</td>
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<td>442.099</td>
<td>124</td>
<td>450.573</td>
<td>163</td>
<td>440.112</td>
</tr>
<tr>
<td>8</td>
<td>455.56</td>
<td>47</td>
<td>446.283</td>
<td>86</td>
<td>459.589</td>
<td>125</td>
<td>461.476</td>
<td>164</td>
<td>444.654</td>
</tr>
</tbody>
</table>
7. Conclusions

After reviewing the previous soil investigating report and pile tests reports, the foundation of the primary digester tank is decided to reanalyze the foundation of tank. Both analytical and numerical analysis has been performed considering conservative and representative values for the soil parameters in the site. The following points are concluded from this study:

- The circular raft and the supporting 193 piles are analyzed as piled-raft rather than pile group and a circular pile cap. This is mainly because the diameter of the raft is much greater than the pile depth. In the piled raft analysis, both the supporting piles and the subgrade soil will share resistance to the applied load of the structure. The main function of piles in the current foundation is to reduce the anticipated settlement of large diameter raft.
- After re-analyzing the pile tests reports by using two well-known methods; Davisson and Brinch-Hansen methods to determine the ultimate carrying capacity of the tested piles. The analysis results and the theoretical calculations have shown that the adopted ultimate and allowable carrying capacity values by the foundation structural designer and by the soil investigation report are overestimated.
- Using SAFE 12 software to analyze the problem numerically, it is found that the anticipated settlement of the raft is less than 100 mm which is within the acceptable limits. It is also found that the piles can support the applied load with a factor of safety of 1.8 for the failed piles and 2.86 for the piles that passed the tests. This value is believed to be converges to more than 2 for both groups of piles if the actual stiffness of each pile is used in the analysis. This is mainly because piles of high stiffness will tend to carry greater load while piles of less stiffness will carry a less load.
8. Funding

This paper is a part of project conducted at the Department of Civil Engineering/University of Baghdad to analysis a foundation of tank in treatment plant. The project funded by Al-Rafideen General Company for Dams.

9. Acknowledgements

The authors expressed their thanks and appreciation to the staff of soil mechanics laboratory at the Department of Civil Engineering/University of Baghdad for their assist in conducting laboratory tests.

10. Conflicts of Interest

The authors declare no conflict of interest.

11. References


Influence of Glass Fibers on Mechanical Properties of Concrete with Recycled Coarse Aggregates

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Received 30 March 2019; Accepted 12 May 2019

Abstract

Despite plain cement concrete presenting inferior performance in tension and adverse environmental impacts, it is the most widely used construction material in the world. Consumption of fibers and recycled coarse aggregates (RCA) can add ductility and sustainability to concrete. In this research, two mix series (100%NCA, and 100%RCA) were prepared using four different dosages of GF (0%GF, 0.25%GF, 0.5%GF, and 0.75%GF by volume fraction). Mechanical properties namely compressive strength, splitting tensile strength, and flexural strength of each concrete mixture was evaluated at the age of 28 days. The results of testing indicated that the addition of GF was very useful in enhancing the split tensile and flexural strength of both RCA and NCA concrete. Compressive strength was not highly sensitive to the addition of GF. The loss in strength that occurred due to the incorporation of RCA was reduced to a large extent upon the inclusion of GF. GF caused significant improvements in the split tensile and flexural strength of RCA concrete. Optimum dosage of GF was determined to be 0.25% for NCA, and 0.5% for RCA concrete respectively, based on the results of combined mechanical performance (MP).

Keywords: Fiber Reinforced Concrete; Recycled Coarse Aggregates; Glass Fibers; Mechanical Properties; Tensile Strength; Flexural Strength.

1. Introduction

Concrete is used more than any other manmade material in the world due to its unique advantages. Formability, high strength (in compression), durability, and the cost-effectiveness of OPC concrete make it more adaptable material than any other conventional material such as wood, steel, bricks, stones, etc. Though concrete has a high compressive strength, but it is brittle and fragile in both tension and bending. Its tensile strength in most of the cases is less than 10% of its compressive strength and typically its tensile strength is neglected in the design of concrete structures [1]. Improving tensile and flexural/bending strength of concrete and minimizing its natural aggregate content can help to add more value to stature of concrete.

To address the lower performance of concrete in tension, different fibers has been used as reinforcement. Fibers of various kinds have been reported to decrease the crack proliferation not only in terms of width but also in numbers when compared to plain concrete. Fibers affect properties of concrete in both fresh and hardened states. Fibers affect workability, strength, ductility, and durability of concrete. But fibers are mainly used to enhance the structural performance of concrete. Fibers have been reported to decrease workability [2], therefore, to maintain workability higher dosages of plasticizers are employed. Various studies have shown that inclusion of fibers improves tensile strength,
flexural strength, toughness, impact resistance, fatigue resistance, the abrasion resistance of concrete [3-5]. Positive effects of fibers on compressive strength have been reported in the studies as about 10% net increase in compressive strength of fiber reinforced concrete was noticed [6-8]. Recommended dosages of most of the fibers vary from 0.5% to 1% in volume fractions [2, 6, 9-12] The reason for the lower volume fractions of fibers being used for the optimal efficiency in increasing the strength properties of concrete is because higher dosages of fibers may result in loss of workability and poor dispersion of fibers. The most common types of fibers which have been used to reinforce concrete and mortars are steel fibers, basalt fibers, glass fibers, polypropylene fibers. Among all steel fibers are the extensively research fibers. Glass fiber (GF) has not been researched to that level, and variances occur in reported results in various studies concerning various parameters of glass fiber reinforced concrete (GFRC). Simes et al. have reported that different kind of fibers may affect the density and porosity of cement mortars differently [13]. They have also reported that polypropylene (PP) fibers decrease the porosity of FRC because their flexibility under compaction cause them to fill voids efficiently. On contrary, GF increase porosity for their rigidity cause them to create voids during the compaction process.

Exploitation of stone quarries for construction aggregates brings a huge negative impact on the environment. Also, natural sources of aggregates are exhausting as demands for them in construction section are growing day by day for new public infrastructure in a developing country like Pakistan. Using recycled aggregates instead of crushed stone aggregates can not only save natural reserves for aggregates but also can save human from issues of waste management pertaining to dumping of construction and demolition wastes. Akhtar et al. [14] reported that the use of recycled aggregates in concrete is eco-efficient and economical than other practices i.e. backfilling, disposal, landfilling, etc.

Use of RCA in concrete as coarse aggregates at very high percentages can affect the strength and durability characteristics of product concrete negatively [14-19]. Coarse aggregates are a major constituent of concrete. Most of the properties of concrete depend on quality of coarse aggregates. RCA usually possess poorer quality than its natural counterpart, due to the presence of low density adhered mortar. Kurda et al. reported in two of their studies that compared to conventional aggregates use of RCA cause insignificant reductions in compressive strength of concrete [15]. They have reported a 10% decrease in compressive strength of concrete when RCA is used as coarse aggregate instead of NCA. Kurad et al. [15] also reported that reductions in global warming potential of concrete are possible if RCA is used in concrete instead of NCA.

Consumption of both fibers (like GF) and RCA may add tensile strength and sustainability to concrete. Impact of GF reinforcement especially on mechanical properties of recycled aggregate concrete has not been studied widely. Rao et al. [20] investigated the behavior of GF reinforced concrete made with 50% RCA using very low fractions of GF. They reported that mechanical performance and ductility of RCA concrete is enhanced at 0.03% GF. But studies pertaining to use of GF with 100% RCA are very scarce. So, the aim of this study was to investigate the mechanical properties of GF reinforced concrete having 100% RCA as coarse aggregates. In this research, three different volume fractions of GF varying from 0.25-0.75% were used in both 100% NCA and 100% RCA concrete mixtures and their strength parameters namely compressive strength, split tensile strength and flexural strength were evaluated and compared.

2. Materials and Methods

2.1. Materials

General purpose (Bestway 43 grade) Portland cement was utilized as binder in this research. This cement follows the specifications for general purpose cement of type II adhering to ASTM C150 [21]. General properties of cement are given in Table 1. Less than 5% weight is retained on 100-micron sieve for this cement.

Natural sand of Lawrence Pur quarry was used as fine aggregate throughout this experimental study. NCA used in this study was crushed limestone obtained from a crushing plant in Margalla Hills, Taxila, Pakistan. RCA was obtained from manual crushing of tested specimen of concrete laboratory of Department of Civil Engineering, University of Engineering and Technology, Taxila, Pakistan. Compressive strength of parent concrete specimens was averaging 30 MPa at 28-days. General properties of both fine and coarse aggregates are listed in Table 2. All the aggregates meet the ASTM C33 requirements of aggregates for concrete [22]. Gradation curves of aggregates are shown in Figure 1, whereas, overview of coarse aggregates (RCA and NCA) is shown in Figure 2.

High range water reducing admixture Sikament 512 was utilized to achieve desired range of workability for all concrete mixes. The triethanolamine and sodium thiocyanate-based plasticizer had the specific weight of 1.12 g/cm³. Admixture meets the requirements of ASTM type F [23]. Potable water was used throughout the research for mixing of concrete.

CEM-Fil 62 chopped strands type of GF has been used in this research it has general properties listed in Table 3. Overview of glass fibers is shown in Figure 3.
Table 1. Chemical and physical properties of cement

<table>
<thead>
<tr>
<th>Chemical properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Calcium Oxide (CaO)</td>
<td>61.72%</td>
</tr>
<tr>
<td>Silica (SiO₂)</td>
<td>21.02%</td>
</tr>
<tr>
<td>Alumina (Al₂O₃)</td>
<td>5.04%</td>
</tr>
<tr>
<td>Iron Oxide (Fe₂O₃)</td>
<td>3.24%</td>
</tr>
<tr>
<td>Magnesium Oxide (MgO)</td>
<td>2.56%</td>
</tr>
<tr>
<td>Sulphur Trioxide (SO₃)</td>
<td>1.51%</td>
</tr>
<tr>
<td>Loss on ignition (LOI)</td>
<td>1.83%</td>
</tr>
<tr>
<td>Insoluble residue (IR)</td>
<td>0.54%</td>
</tr>
<tr>
<td>Free lime</td>
<td>0.98%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Physical properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>3.12</td>
</tr>
<tr>
<td>Specific surface (cm²/g)</td>
<td>3720</td>
</tr>
<tr>
<td>Setting time (mins) (Initial)</td>
<td>102</td>
</tr>
<tr>
<td>Setting time (mins) (Final)</td>
<td>608</td>
</tr>
<tr>
<td>Strength at 28 days (MPa)</td>
<td>41.45</td>
</tr>
</tbody>
</table>

Table 2. Properties of aggregates

<table>
<thead>
<tr>
<th>Property</th>
<th>Sand</th>
<th>NCA</th>
<th>RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. nominal size (mm)</td>
<td>4.75</td>
<td>12.50</td>
<td>12.50</td>
</tr>
<tr>
<td>Min. nominal size (mm)</td>
<td>0.075</td>
<td>4.75</td>
<td>4.75</td>
</tr>
<tr>
<td>Particle density (g/cm³)</td>
<td>2.68</td>
<td>2.71</td>
<td>2.39</td>
</tr>
<tr>
<td>Saturated Surface dry (SSD) water absorption (%)</td>
<td>0.89</td>
<td>1.45</td>
<td>8.65</td>
</tr>
<tr>
<td>10% fine value (kN)</td>
<td>-</td>
<td>157</td>
<td>125</td>
</tr>
<tr>
<td>Dry rodded density (kg/m³)</td>
<td>1624</td>
<td>1547</td>
<td>1271</td>
</tr>
</tbody>
</table>

Figure 1. Gradation curves of all aggregates
2.2. Composition of Concrete Mixtures

Two series of concrete mixes were prepared. All concrete mixtures were designed for cylindrical compressive strength of 30 MPa following ACI guidelines. The first series was prepared using NCA as coarse aggregates and the second series had RCA as coarse aggregate. Both series had a total of four-member mixtures. The first mixture in each series aid as a control for other member mixtures which are fiber reinforced with GF. Fiber reinforced mixtures has three different dosages of GF namely 0.25%, 0.5% and 0.75% by volume fractions. Details of each mix are shown in Table 4. As loss in workability of FRC mixes compared to the plain concrete mix is inevitable, thus dosage of plasticizer was varied to achieve a slump in the range of 80-100 mm.
### Table 4. Mix proportions in unit cubic meter of each concrete mixture

<table>
<thead>
<tr>
<th>Mix Series</th>
<th>Type of Coarse aggregate</th>
<th>GF by volume (%)</th>
<th>Cement (kg/m³)</th>
<th>Medium sand (kg/m³)</th>
<th>NCA (kg/m³)</th>
<th>RCA (kg/m³)</th>
<th>GF (kg/m³)</th>
<th>Admixture (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Additional water (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>NCA</td>
<td>0</td>
<td>390</td>
<td>845</td>
<td>880</td>
<td>0</td>
<td>0</td>
<td>215</td>
<td>6.38</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>390</td>
<td>845</td>
<td>880</td>
<td>0</td>
<td>6.5</td>
<td>0.45</td>
<td>215</td>
<td>6.38</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
<td>390</td>
<td>845</td>
<td>880</td>
<td>13</td>
<td>0.98</td>
<td>215</td>
<td>6.38</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.75</td>
<td>390</td>
<td>845</td>
<td>880</td>
<td>19.5</td>
<td>2.13</td>
<td>215</td>
<td>6.38</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>RCA</td>
<td>0</td>
<td>390</td>
<td>845</td>
<td>0</td>
<td>725</td>
<td>0.24</td>
<td>6.5</td>
<td>215</td>
<td>56.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>390</td>
<td>845</td>
<td>0</td>
<td>725</td>
<td>6.5</td>
<td>0.78</td>
<td>215</td>
<td>56.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
<td>390</td>
<td>845</td>
<td>0</td>
<td>725</td>
<td>13</td>
<td>1.45</td>
<td>215</td>
<td>56.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.75</td>
<td>390</td>
<td>845</td>
<td>0</td>
<td>725</td>
<td>19.5</td>
<td>2.68</td>
<td>215</td>
<td>56.44</td>
</tr>
</tbody>
</table>

### 2.3. Preparation and Testing of Specimens

Mixing of all concrete mixtures was done in a mechanical mixer of 0.15 m³ capacity. First aggregates were mixed with 2/3 of water for about 6 mins, to allow aggregates sufficient time to absorb water up to their 80% capacity [24], then cement and GF (in case of fiber reinforced mixes) were added with remaining 1/3 of water and mixing continued for about 4 mins. Required dosage (already determined in trials) of plasticizer was also added along with 1/3 of water in case of the mixtures which required plasticizer to achieve the desired range of workability (80-100 mm slump).

Slump test was used to determine the workability of fresh concrete according to ASTM C143 [25]. 100 mm cubes were cast to measure compressive strength of concrete mixtures following BS: EN 12390-3 [26]. Splitting tensile strength test was conducted on cylindrical specimens of 150 mm diameter x 300 mm height to estimate indirect tensile strength of concrete specimen according to ASTM C496 [27]. Three-point bending test on prisms of dimensions 100 mm x 100 mm x 500 mm was conducted to estimate flexural strength of each mixture according to ASTM C78 [28]. All specimens after casting were kept for setting in molds for about 24 hours. After demolding all specimens were cured in a water tank for the age of 28 days. To determine each strength parameter pertaining to a particular mix three replicate specimens were tested and their average was reported in this research paper. Moreover, the schedule of testing is shown in Table 5. Overview of casting and experimental test setups are shown in Figures 4 and 5 respectively.

### Table 5. Schedule of testing

<table>
<thead>
<tr>
<th>Mix Series</th>
<th>Type of coarse aggregate</th>
<th>GF (%)</th>
<th>Compression testing BS: EN 12390-3 [26]</th>
<th>Split tensile testing ASTM C496 [27]</th>
<th>Flexural testing ASTM C78 [28]</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>NCA</td>
<td>0</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.75</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>II</td>
<td>RCA</td>
<td>0</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.75</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

To optimize the concrete mixes based on their mechanical performance various weights were assigned to each of the strength parameters according to its importance in the design of concrete. Highest weight (of 4) was considered for compressive strength followed by flexural strength (of 2) and split tensile strength (of 1). Mechanical performance of each mixture was evaluated by using Equation 1. In Equation 1, the term ‘f’ refers to strength, ‘MIX’ refers to a particular mix whose mechanical performance (MP) is being evaluated, and ‘CON’ refers to plain NCA mix having 0%GF.

\[
MP(\%) = \frac{4 \times f_{\text{compressive MIX}} + 2 \times f_{\text{flexural MIX}} + f_{\text{split MIX}}}{7} \times 100
\]
3. Results and Discussions

3.1. Compressive Strength

Results of compression testing are shown in Figure 4. Relative analysis of compressive strength results is also presented in Table 5. General trend shows that inclusion of RCA was detrimental to the compressive strength of concrete. GF inclusion improved the compressive strength of concrete by trivial margins compared to the plain concrete mixes. Compared to NCA mix compressive strength reduced by about 12% when RCA was used as coarse aggregate. As RCA contain some adhered mortar inherited from their parent concrete cause the increase in global porosity of concrete which subsequently reduces compressive strength of concrete [15]. Another factor which may contribute to reductions in compressive strength is that RCA mixes require more water than NCA mixes.

The inclusion of GF caused a trivial increase in compressive strength of NCA concrete. At 0.25% dosage of GF net increase in compressive strength of about 9.7% was observed, see Table 5. Compressive strength did not significantly improve compared to plain NCA mix on further incorporation of GF beyond 0.25%. Although GF had higher specific weight than cement matrix under compaction, their movements can generate voids in concrete due to their rigidity, which may increase the porosity [13]. This can be blamed to decrease compressive strength at dosages higher than 0.25%. Although compressive strength of mixes with GF higher than 0.25% showed a decreasing trend but outperformed plain NCA mix at all dosages. High et al. [9] reported a small increase in compressive strength upon the inclusion of basalt fibers, similarly, Kizilkanat et al. [29] and Song et al. [6] reported an insignificant increase in compressive strength concrete mixes at the age of 28 days upon the inclusion of different types of fibers compared to the control plain concrete mixes.
GF addition in RCA mixes also showed ordinary improvements in compressive strength. A small increase of about 8.9% was noticed at 0.5% GF when compared to the plain RCA mix. Upon further inclusion of GF did not show any improvements in compressive strength. The inclusion of GF sufficiently compensates the loss in strength of RCA mixes compared to plain NCA mixes, see Table 6. RCA mix with 0.5% GF performs almost up to the 96% potential of plain NCA mix. It can be said the GF inclusion can help in recovering the drop in compressive strength of concrete with RCA as coarse aggregates.

<table>
<thead>
<tr>
<th>Dosage of GF</th>
<th>NCA mixes</th>
<th>RCA mix</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Relative compressive strength w.r.t plain NCA mix</td>
<td>Relative compressive strength w.r.t plain RCA mix</td>
</tr>
<tr>
<td>0.00%</td>
<td>100.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>0.25%</td>
<td>109.7%</td>
<td>105.8%</td>
</tr>
<tr>
<td>0.50%</td>
<td>108.5%</td>
<td>108.9%</td>
</tr>
<tr>
<td>0.75%</td>
<td>106.5%</td>
<td>105.0%</td>
</tr>
</tbody>
</table>

### 3.2. Split Tensile Strength

Split tensile strength does not represent the true tensile strength of concrete, but it is a better estimation of the tensile strength and ductility of concrete, as the specimen is allowed to fail under loads that almost split a sample into halves. A brittle specimen would fail suddenly under splitting action of loads after the appearance of the first crack, but a ductile specimen undergoes a failure gradually after the first crack (with enough warning). So, split tensile test gives a good idea about the ductility of a specimen.

Results of splitting tensile strength of all concrete mixes are shown in Figure 7. Whereas, relative analysis of results of split tensile strength are presented in Table 7. While testing it was observed that fiber-reinforced specimens did take sufficient load after the appearance of the first crack, whereas, plain concrete specimens went under failure quickly after the appearance of cracks. RCA inclusion affects the split tensile strength as badly as it did compressive strength. The split tensile strength of plain RCA mix reduced about 11% when compared to that of the plain NCA mix. It can be seen in Figure 7 that effect of GF inclusion was more useful on the split tensile strength of concrete. At the dosage of 0.5%GF, both RCA and NCA mixes showed maximum tensile strengths compared to their corresponding plain concrete mixes.
The tensile strength of NCA mixes increased by about 13%, 18% and 16% at 0.25%, 0.5% and 0.75% volume fractions of GF respectively when compared with plain NCA mix. Similarly, split tensile strength of RCA mixes increased by about 18%, 22%, and 20% at 0.25%, 0.50%, and 0.75% dosages of GF respectively. As GF have higher tensile strength about 1700 MPa, they increase the stiffness of cement matrix of concrete against tensile forces. Under splitting action of loads, both cohesion of cement matrix and glass fibers offer higher resistance to cracks. In plain concrete mixes where tensile strength only depends on the cement matrix show lesser strength than fiber-reinforced concrete. At dosage higher than 0.5% split tensile strength did not improve which may be ascribed to difficulty in the dispersion of fibers as noted by researchers [29]. Jiang et al [3] reported that inclusion of basalt fiber at optimum dosage increased split tensile strength by about 20%.

Relative analysis of RCA mixes with respect to plain NCA mix presented in Table 7. It shows that RCA mixes with GF outperform plain NCA mix with significant margin. Unlike the results of compressive strength, RCA mixes with GF perform better than plain NCA concrete under tensile load. It is worth mentioning here that increasing the dosage of GF beyond 0.25% split tensile strength of both NCA and RCA concrete mixes does not undergo a marginal increase.

Table 7. Relative analysis of results of split tensile strength

<table>
<thead>
<tr>
<th>Dosage of GF</th>
<th>NCA mixes</th>
<th>RCA mixes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Relative split tensile strength w.r.t plain NCA mix</td>
<td>Relative split tensile strength w.r.t plain RCA mix</td>
</tr>
<tr>
<td>0%</td>
<td>100.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>0.25%</td>
<td>113.7%</td>
<td>117.9%</td>
</tr>
<tr>
<td>0.50%</td>
<td>118.2%</td>
<td>122.6%</td>
</tr>
<tr>
<td>0.75%</td>
<td>116.2%</td>
<td>120.1%</td>
</tr>
</tbody>
</table>

### 3.3. Flexural Strength

Flexural strength of concrete shows its ability to resist bending loads. In plain concrete flexural strength mainly depends on the strength of the bond between constituents of concrete. It is very well known that harshness and grading of aggregates play a major role in defining flexural strength of plain. Well graded and harsh aggregates offer better flexural strength properties owing to better aggregate interlock and increased internal-friction. In the fiber-reinforced matrix, fibers by virtue of their higher tensile strength increase rigidity of the cement matrix of concrete hence higher flexural strength can be anticipated in fiber-reinforced concrete than that of the plain concrete.
Results of flexural testing are shown in Figure 8, whereas relative analysis of results of flexural strength is presented in Table 8. While testing prismatic specimens under three-point loading, it was observed that fiber reinforced specimens did take load after the appearance of crack at the bottom of prisms and failure was gradual, whereas, in case of plain concrete specimens’ failure was sudden. It can be seen from Figure 8 that for both NCA and RCA mixes optimum dosage of GF is 0.25%, increasing GF beyond this dosage does not improve the flexural strength of concrete.

NCA mixes experienced a useful boost in flexural strength compared to the corresponding plain concrete mix. Flexural strength increased by about 28%, 24%, 22% at 0.25%, 0.5% and 0.75% dosages of GF respectively. The results are in accordance with studies of [6, 29, 30]. Jiang et al. [3] also reported a drop in flexural strength when GF dosage increased beyond 0.25%, maximum flexural strength was mentioned at 0.3% dosage of GF. They mentioned that higher dosages of GF were ineffective compared its lower dosages due to dispersion issues.

Table 8. Relative analysis of results of flexural strength

<table>
<thead>
<tr>
<th>Dosage of GF</th>
<th>NCA mixes</th>
<th>RCA mixes</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>100.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>0.25%</td>
<td>128.2%</td>
<td>134.3%</td>
</tr>
<tr>
<td>0.50%</td>
<td>124.9%</td>
<td>131.9%</td>
</tr>
<tr>
<td>0.75%</td>
<td>122.6%</td>
<td>128.0%</td>
</tr>
</tbody>
</table>

Fiber reinforced RCA mixes undergone a significant increase in flexural strength compared to the corresponding plain mix. At an optimum dosage of GF (0.25%) flexural strength of RCA concrete compared to the plain RCA was increased by more than 34%. This increase was dropped to 28% at 0.75% GF. All RCA mixes with GF showed higher strengths than the corresponding plain RCA mix. Compared to the plain NCA mix flexural strength of RCA concrete was about 23% higher at an optimum dosage of GF.

Strength parameters of fiber-reinforced mixes confirm that addition of GF has been more useful to flexural strength than both splits tensile and compressive strength. Addition of GF on RCA helped in recovering loss in flexural strength to a great degree.

### 3.4. Mechanical Performance (MP)

MP (%) of each mix using Equation 1 was calculated by incorporating results of compressive, split tensile, and flexural strength. MP of each mix is shown in Figure 9. It can be observed that all mixes with GF outperform plain NCA mix (CON) in MP. For fiber-reinforced NCA concrete mixes, maximum MP is at the dosage of 0.25%, whereas for fiber-reinforced RCA mixes maximum MP is at the dosage of 0.5%. This is due to the fact that RCA mixes showed
higher values of compressive and split tensile strength at the dosage of 0.5%. Considering MP (%) RCA mix with 0.5% GF at the optimum dosage outperforms conventional plain NCA mix by more than 5%.

RCA mixes achieve sufficient MP at 0.25% of GF compared to their plain mix, increasing the dosage of GF to 0.5% cause a further increase of 2% in MP, therefore, 0.25% dosage of GF can also be considered for optimum both NCA and RCA mix from an economic point of view. Increasing the dosage of GF from 0.25% to 0.5% would increase the quantity of GF in 1 m3 of concrete from 6.5 kg to 13 kg. Out of all ingredients, GF is the most expensive (nearly its market price is 7 USD/kg). So, 0.25% dosage of GF can be considered optimum. Also, a higher dosage of GF would increase the dosage of plasticizer which is also considered very expensive (nearly its market price is 1 USD/kg).

GF inclusion in concrete is more useful to tensile strength properties of concrete i.e. split tensile and flexural strength, it can be seen in Figure 10, that ratio of split tensile strength and compressive strength is higher for the fiber reinforced mixtures than that of the plain mixes (having 0%GF). For example, for fiber-reinforced concrete mixes split tensile strength is approximately 9% of the compressive strength, whereas, for plain concrete mixes split tensile strength is approximately 8% of that of the compressive strength. Similarly, the ratio of flexural strength and compressive strength as shown in Figure 11, is higher for the fiber-reinforced concrete mixes. For example, for fiber-reinforced concrete mixes, flexural strength is approximately 12% of the compressive strength, whereas, for plain concrete mixes flexural strength is approximately 9.5% of that of the compressive strength. It can be concluded that addition fiber addition increases the ratio of tensile-to-compressive strength of concrete.
4. Conclusions

Following conclusions can be drawn from this research paper:

- RCA and GF inclusion influence the workability of concrete badly and both increase the demand of plasticizer to achieve the desired range of slump.
- RCA influence the strength parameters of concrete badly. When RCA replaces NCA it reduces the compressive strength, split tensile strength, and flexural strength by about 12%, 11%, and 8% respectively.
- Influence of GF in both NCA and RCA mixes was more useful on flexural and split tensile strength of concrete than that of the compressive strength.
- Although GF improves the compressive strength of RCA concrete but it does not outperform plain NCA concrete at the optimum dosage (0.5% of GF). RCA concrete with 0.25-0.5% of GF outperforms plain NCA concrete in split tensile and flexural strength test.
- Combined MP indicate that RCA concrete with 0.25% GF can outperform plain NCA concrete by a fair difference mainly due to boost in split tensile and flexural strength.
- Optimum dosage of GF considering combined MP is 0.25% for NCA concrete and 0.50% for RCA concrete. But considering both economy and MP optimum dosage of GF may be taken as 0.25% for both NCA and RCA concrete mixes.

5. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Economic and Environmental Impacts of Cropping Pattern Elements Using Systems Dynamics

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Received 22 December 2018; Accepted 17 April 2019

Abstract

Tragedies arising from poor water resources management and planning are significantly more relevant than climate change and frequent natural droughts, especially in arid and semi-arid areas. Nearly 92% of total water is allocated to the agricultural sector in Iran. In this situation, cultivation patterns play an important role in agricultural water management. Evaluating the effect of each crop would help the stakeholders make a rational decision in choosing appropriate cropping patterns to avoid groundwater depletion as well as maintain their livelihoods. The Qazvin plain in Iran, whose aquifer has had a drawdown of nearly 20m during the last 15 years, was used in this case study. It has been modeled using system dynamics, which includes two subsystems: hydrology, for calculating groundwater level, and economy, for defining farmer’s income in the years from 1997 to 2011. The system dynamics, which included 17 crops, was developed after calibration by simple genetic algorithm and verification under extreme condition tests. To identify the economic and environmental effect of each of the crops, the system dynamics was run 18 times, removing crops one by one. It has been found that wheat plays an important role in causing a negative water balance but does not affect the farmers’ incomes as significantly as grapes. Two indicators, which included sustainable water resources and water exploitation, were employed to assess the scenarios as well. According to the results, no scenarios are fully sustainable for maintaining a steady aquifer, but scenario 1, which removed wheat from the cropping pattern, is the most sustainable and puts the least pressure on the aquifer.

Keywords: Groundwater Level; System Dynamics; Farmers’ Income; Sustainability; Wheat.

1. Introduction

Recently, increasing population, climate change, industrialization, urbanism, etc. have been affecting water resources—especially aquifers. Very often in arid and semi-arid countries, farmers use natural resources unwisely in order to survive. This kind of behavior causes a crisis in job security, drinking demand and food security. The long-term decreasing groundwater level has resulted from many reasons such as weak policy, drought, inappropriate management and planning of water resources. Since both groundwater level and farmers’ income are affected by cropping pattern, defining an appropriate cropping pattern is an imperative issue that should be noted [1–4]. Consequently, both decreasing groundwater depletion and increasing the farmers’ income are essential considerations [4–7]. Cropping pattern could be
impacted by many factors such as policy change, importing and exporting rules of agricultural products, climate, product demands, water supply, job opportunities and so on.

During recent years, a large number of studies, many of which are based on cropping pattern optimization, have been done [2, 5, 6, 8–12]. Fazlali et al. [13] used a coupled simulation-optimization model to determine the optimal cropping pattern in the Arayez plain in the Karkheh river basin in Iran. By integration of Network Flow Programming (NFP) as simulation model, and the Shuffle Frog Leaping Algorithm (SFLA) as optimization model, the total net benefit gained from crop production was maximized. The results of the coupled SFLA-NFP model show that the net benefit increases 12% compared to the present situation in the plain. Jebelli et al. [14] developed a model to maximize farmers’ gross income by optimizing the cropping pattern while satisfying all of the imposed constraints in the Tigray region in Ethiopia. Its constraints included water demand, crop disease, and pest resistance, market price, level of fertilizer input, intensity of labor requirement, capital requirements, and post-harvest processing requirements. The results showed that the percentages of only two crops; tomato-from 5% to 8%, and barley-from 3% to 44%, were increased in the optimal cropping pattern. Osama et al. [15] developed a linear programming to optimize land allocation in order to maximize the net annual income of three old areas of Egypt. In the model, different constraints such as water availability, land availability in different seasons of the year, self-sufficiency ratios, and actual areas of crops under existing patterns of cropping were applied to optimize the cropland of 28 crops in the years from 2008 to 2012. The results showed that the benefits are increased by an average of 6.66% ±0.84 over the modeling period. Varade et al. [8] defined a multi-objective optimization problem to determine optimal cropping pattern for maximizing the net annual returns and conserve groundwater resources simultaneously. The cropping patterns of 33 crops were optimized using two PSO and Jaya algorithms and the results led to higher income and less water allocation compared to the existing cropping pattern. Youse et al. [16] developed a model that included three objective functions that would maximize the benefits, reduce nitrogen leaching, and improve the rate of aquifer recharge, both together and separate. In the developed model, Particle Swarm Optimization (PSO) integrated with an additive weighting method and a Multi-Objective Particle Swarm Optimization (MOPSO) algorithm were used to find the optimal cropping. The results showed that it is possible to increase water efficiency; while increasing farmers’ benefits, and decreasing nitrogen leaching. Rath et al. [17] identified a suitable cropping pattern through optimization techniques such as LINDO and Genetic Algorithm. The developed cropping pattern gave the net benefit of about 46% more than the present habit.

The literature review shows previous studies did not mention the impacts of each single crop of the cropping pattern on the farmers’ income and groundwater level, nor did they focus on over time. This research is concerned with evaluating the effect of each crop on groundwater drawdown and the farmers’ income simultaneously over the same period.

2. Materials and Methods

2.1. Case Study

The Qazvin aquifer, in Iran, is one that has a negative balance. As figure 1 shows, the average yearly decrease in groundwater level is 1.3 m. The aquifer is the most important resource in the Qazvin plain for all demands such as drinking, industry, and agriculture. Farmers are the main groundwater users, by 1200 MCM. Additionally, the income of most people in the Qazvin plain is extremely dependent on agriculture. Therefore, the aquifer plays an important role in the economy of this area.

![Figure 1. History of groundwater levels over 15 years](Image)

Figure 2 indicates the location of the Qazvin plain, its climate classification as well as its water resources. The Qazvin plain’s needs are provided by transfer water from Taleghan dam, some rivers such as Khar Rood, Abhar Rood, and Haji
Arab Rood and its aquifer. To model the case study, the system dynamics was used as an adequate tool. The system dynamics of the Qazvin plain has two subsystems including hydrology and economy, which will be described in the next section.

![Figure 2. Location map of case study](image)

Following flowchart shows the steps that have been taken in doing the study (Figure 3). Each stage of the research is explained in detail as follows.

-Develop a SD including two subsystems: hydrology, and economy
- Calibrate the SD using a Simple Genetic Algorithm (SGA) to minimize the residuals
- Run the SD for the actual condition with 17 crops (Scenario 0)
- Design Scenarios
  For crop $i = 1:17$
  - Remove crop $i$ (scenario $i^{th}$)
  - Run the new SD with 16 crops
  - Calculate the Farmers’ Income Deficiency (FID %) and the Avoiding Drawdown (ADW %) by removing crop $i$ over 15 years
  - Calculate average annual SUI and WEI+

Figure 3. Flow chart of research methodology

2.2. System Dynamics

System dynamics (SD) is an approach to understanding the nonlinear behavior of complex systems over time using stocks, flows, internal feedback loops, and time delays [18–20]. Additionally, it makes it possible for us to model action and reaction of physical and non-physical factors on each other as if we can evaluate the reaction of the extraction of the aquifer on employment and so on. The Qazvin plain is modeled using two subsystems: hydrology, and economy.

2.2.1. Hydrology Subsystem

The subsystem has two main stock variables including surface water and groundwater.
2.2.1.1. Groundwater

The Qazvin plain has the main aquifer located in the middle of the plain. The aquifer has an important role in growing agriculture and it is the most important water resource in the plain as well. Therefore, conserving the aquifer is essential to guaranteeing appropriate living conditions in the area for the future.

The groundwater volume is the stock variable of the subsystem that is calculated by subtracting inflow from outflow (Figure 4). The groundwater volume in each year and cumulative groundwater volume was computed using Equations 1 and 2 respectively.

\[
\Delta S_{G,t} = Q_{GI,t} - Q_{Go,t} \tag{1}
\]

\[
S_{G,t} = \Delta S_{G,t} + S_{G,t-1} \tag{2}
\]

Where, \(\Delta S_{G,t}\) is the change in groundwater storage in year \(t\) (MCM), \(Q_{GI,t}\) is the amount of inflow into the aquifer in year \(t\) (MCM), \(Q_{Go,t}\) is the amount of outflow from the aquifer in year \(t\) (MCM), \(S_{G,t}\) is the cumulative groundwater volume in the aquifer until year \(t\) (MCM), and \(S_{G,t-1}\) is the cumulative groundwater volume until year \(t-1\) (MCM).

![Figure 4. Flow diagram of groundwater stock](image)

Then, the amount of inflow and outflow of the aquifer are calculated by Equations 3 and 4.

\[
Q_{GI,t} = D_{agrilt} \times RC_{agrilt} + S_R + Re_{TD,t} + Re_{R,t} + IF_{an,G} + (D_I + D_{Dr}) \times RC_{I,DR} \tag{3}
\]

\[
Q_{Go,t} = D_I + D_{Dr} + D_{agrilt} + E_G + OF_G + Dr_R \tag{4}
\]

Where, \(RC_{agrilt}\) is the return flow coefficient to estimate the return water from irrigation, \(S_R\) is the amount of seepage from the river (MCM), \(Re_{TD,t}\) is the amount of artificial recharge from Taleghan dam in year \(t\) (MCM), \(Re_{R,t}\) is the amount of precipitation recharge in year \(t\) (MCM), \(IF_{an,G}\) is annual average recharge by other aquifers (MCM), \(D_I\) is the average annual urban demand (MCM), \(D_{Dr}\) is the average annual industrial demand (MCM), \(RC_{I,DR}\) is the return coefficient from extracted water usage for urban and industry needs, \(D_{agrilt}\) is the amount of water that was allocated to industry and urban demand (MCM), \(E_G\) is the amount of evaporation from the aquifer (MCM), \(OF_G\) is the annual average recharge from other aquifers nearby (MCM), and \(Dr_R\) is the amount of groundwater discharge to river (MCM).

Then, the amount of the equivalent groundwater level in year \(t\) is computed using Eq. 5.

\[
GW_{l,t} = \frac{S_{G,t}}{A_{aq} \times S} \tag{5}
\]

Where, \(GW_{l,t}\) is the groundwater level in year \(t\), \(S\) is the coefficient storage of the aquifer, and \(A_{aq}\) is the area of the aquifer.
2.2.1.2. Surface Water

The most important surface water resource in the plain comes from Taleghan dam. Nearly 10% of it is allocated to recharge the aquifer using recharge wells and the remains feed the irrigation network which is located in the northern part of the plain (Figure 2).

Because there is not a reservoir in the plain, changes in storage should be zero (Eq.6). Some water is lost through evaporation and flow into other watersheds, while the rest is consumed by the irrigation network (Figure 5).

![Figure 5. Flow diagram of surface water stock](image)

$$Q_{SL,t} = Q_{So,t}$$  \hspace{1cm} (6)

Where, $Q_{SL,t}$ and $Q_{So,t}$ are the amount of inflow and outflow surface water respectively which are calculated using Equations 7 and 8.

$$Q_{SL,t} = AI_{Ta,t} \times 0.9 + RF_t + RI$$  \hspace{1cm} (7)

$$Q_{So,t} = OF_S + E_S + D_{agr,t}^S$$  \hspace{1cm} (8)

Where, $AI_{Ta,t}$ is the allocation from Taleghan dam into the irrigation network in year $t$ (MCM), $RF_t$ is the amount of the runoff in year $t$ (MCM), $RI$ is the inflow from the main rivers into the plain (MCM), $OF_S$ is the surface water outflow from the plain (MCM), $E_S$ is the amount of the evaporation (MCM), and $D_{agr,t}^S$ is the amount of the surface water used for gross irrigation demand (MCM).

2.2.2. Economic Subsystem

Seventeen crops that occupy most cropland in the Qazvin plain were introduced to SD. In this subsystem, yields and water consumption of crops were assigned as a function of cropland, unit yields and unit water demand (Figure 6). Water consumption of each crop was determined using Equation 9 and historical data that was collected by the agricultural office.

$$WROC^Z_{t} = AWROC_Z \times CL^Z_{t}$$  \hspace{1cm} (9)

Where, $WROC^Z_{t}$ is the total net water consumption of crop $Z$ in year $t$ (MCM), $AWROC_Z$ is the net average water consumption of crop $z$ in the Qazvin plain, and $CL^Z_{t}$ is the cropland of crop $Z$ in year $t$. Then, the total gross water requirement of all crops was computed by Equation 10.

$$D_{agr,t} = \sum WROC^Z_{t} \times \frac{1}{EP}$$  \hspace{1cm} (10)
Where, $D_{agri,t}$ and $EP$ are the total gross irrigation water needing in year $t$ and the irrigation system efficiency respectively.

The total farmers’ net income during the studied period was computed using Equations 11 and 12.

$$TNI_{net}^t = NI_{net}^t + TNI_{net}^{t-1}$$  \hspace{1cm} (11)

$$NI_{net}^t = GI_{total}^t - \sum Y_z^t \times C_z$$  \hspace{1cm} (12)

Where, $TNI_{net}^t$ is the farmers’ total net income until year $t$ ($\$), $NI_{net}^t$ is the farmer’s net income in year $t$ ($\$) calculated using Eq.12, $TNI_{net}^{t-1}$ is the farmers’ total net income until year $t$ ($\$), $GI_{total}^t$ is the total gross farmers’ income in year $t$ ($\$), $Y_Z^t$ is the total production of crop $Z$ in year $t$ (ton), and $C_z$ is the cost of crop production $z$ in dollar per ton.

To calculate the amount of products in each year (Equation 13), historical data was used, which shows the amount of yield per hectare of each crop in the Qazvin plain.

$$Y_Z^t = \text{Unit}_Y Z^t \times CL_Z^t$$  \hspace{1cm} (13)

Where, $\text{Unit}_Y Z^t$ is the crop yield of crop $z$ in year $t$ (ton).

The gross income of the farmers was calculated in year $t$ using Equation 14.

$$GI_{total}^t = \sum Y_z^t \times Pr_z^t$$  \hspace{1cm} (14)

Where, $Pr_z^t$ is the price of crop $Z$ in year $t$ ($\$).

![Flow diagram of the economic subsystem](image)

**Figure 6.** Flow diagram of the economic subsystem

### 2.3. Calibration of Model

Because of variety of crops that are grown in the case study that occupied less cropland but didn't consider in the model the SD model was calibrated to meet the historical data. To do this, a Simple Genetic Algorithm (SGA) as an optimization model was employed. The objective function of SGA (Eq. 15) was defined in such a way that leads to reducing the difference between simulated and observed groundwater levels from 1997 to 2011.

$$\text{minimize} \sum_{i=1}^{15} (GI_{obs}^t - GI_{SD}^t)^2$$  \hspace{1cm} (15)
Where, \( GL_{Ob}^t \) and \( GL_{SI}^t \) are the observed and simulated groundwater level in year \( t \) respectively.

### 2.4. Scenarios

To evaluate the effect of each crop on the water resources and farmers’ income, by eliminating crops one by one, different scenarios were defined (Table 1). In each scenario, while one crop was eliminated and others remained unchanged, groundwater levels and the farmers’ total net income over fifteen years were calculated.

Farmers’ income, based on the historical data, was considered as farmers’ potential income, therefore the deficit of farmers’ income could be calculated in each scenario. Using groundwater level as the environmental factor in each scenario, scenario 0 was used as a benchmark. Then the amount of avoiding drawdown and the percentage of the income deficiency were calculated using Equations 16 and 17.

\[
ADW = \frac{DW_{max} - DW_{SCI}}{DW_{max}} \times 100
\]

\[
FID = \frac{FI_{max} - FI_{SCI}}{FI_{max}} \times 100
\]

Where, \( ADW \) is the percentage of avoiding drawdown (\%), \( DW_{max} \) is the maximum drawdown, which is resulted during fifteen years (m), \( DW_{SCI} \) is the drawdown that would arise from scenario \( i \) (m), \( FID \) is the percentage of the Farmers’ Income Deficiency (\%), \( FI_{max} \) is the maximum farmer’s net income, which farmer earned during 15 years (billion dollars), and \( FI_{SCI} \) is the farmers’ net income that would be gained in scenario \( i \) (billion dollars).

### Table 1. Detail of scenarios

<table>
<thead>
<tr>
<th>No.</th>
<th>Name of scenario</th>
<th>Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Scenario 0</td>
<td>historical cropping pattern</td>
</tr>
<tr>
<td>2</td>
<td>Scenario 1</td>
<td>Remove wheat</td>
</tr>
<tr>
<td>3</td>
<td>Scenario 2</td>
<td>Remove barley</td>
</tr>
<tr>
<td>4</td>
<td>Scenario 3</td>
<td>Remove potato</td>
</tr>
<tr>
<td>5</td>
<td>Scenario 4</td>
<td>Remove tomato</td>
</tr>
<tr>
<td>6</td>
<td>Scenario 5</td>
<td>Remove maize</td>
</tr>
<tr>
<td>7</td>
<td>Scenario 6</td>
<td>Remove silage</td>
</tr>
<tr>
<td>8</td>
<td>Scenario 7</td>
<td>Remove cucumber</td>
</tr>
<tr>
<td>9</td>
<td>Scenario 8</td>
<td>Remove rapeseed</td>
</tr>
<tr>
<td>10</td>
<td>Scenario 9</td>
<td>Remove sugar beet</td>
</tr>
<tr>
<td>11</td>
<td>Scenario 10</td>
<td>Remove onion</td>
</tr>
<tr>
<td>12</td>
<td>Scenario 11</td>
<td>Remove cotton</td>
</tr>
<tr>
<td>13</td>
<td>Scenario 12</td>
<td>Remove melon and watermelon</td>
</tr>
<tr>
<td>14</td>
<td>Scenario 13</td>
<td>Remove alfalfa</td>
</tr>
<tr>
<td>15</td>
<td>Scenario 14</td>
<td>Remove peach and nectarine</td>
</tr>
<tr>
<td>16</td>
<td>Scenario 15</td>
<td>Remove grapes</td>
</tr>
<tr>
<td>17</td>
<td>Scenario 16</td>
<td>Remove apple</td>
</tr>
<tr>
<td>18</td>
<td>Scenario 17</td>
<td>Remove pistachios</td>
</tr>
</tbody>
</table>

### 2.5. Evaluating Indicator

To evaluate the impact of each scenario on the aquifer and farmers’ income two indicators were developed, Sustainable Index (SUI) and Water Exploitation Index (WEI+).
2.5.1. Sustainable Index (SUI)

To quantify the sustainability of water resources systems in each scenario as well as evaluate and compare them with each other this index was employed which is calculated using Equation 18 [22]. A negative value of SUI indicates the extra water usage and a positive value indicates the proper water abstraction strategies.

\[
SUI = \frac{Q_{GI} - D_{agri}^G - D_t - D_{Dr}}{Q_{GI}}
\]  

(18)

Where, SUI is the sustainable index, \( Q_{GI} \) is the average annual recharge (MCM), \( D_{agri}^G \), \( D_{Dr} \) and \( D_t \) are the average annual agriculture, urban and industrial demand respectively (MCM).

2.5.2. Water Exploitation Index (WEI+)

The index represents the pressure of water abstraction on the available freshwater resources [22]. The index could be defined as the ratio of average annual demands into average recharge. To solve the uncertainty in the assessment of demands and water resources values, a modified water exploitation index (Equation 19) called WEI+ has been developed [23]. The near zero value shows less pressure on the aquifer.

\[
WEI^+ = \frac{D_{agri}^G + D_t + D_{Dr} - R_{UW}}{Q_{GI} - R_{UW}} \times 100
\]  

(19)

Where, \( WEI^+ \) is the modified water exploitation index (%), and \( R_{UW} \) is the total return water (MCM).

3. Results and Discussion

3.1. Model Calibration

To check the accuracy of the SD model that was developed in Matlab, all simulated stock variables were compared with the values that were computed using Vensim over the simulated period. Then, our model was calibrated by a simple genetic algorithm to reduce error between the simulated and measured data. Table 2 shows the statistical indicators before and after calibrating. As the data shows, the objective function has a mean square error of 3.07 after calibration.

<table>
<thead>
<tr>
<th>Statistical index</th>
<th>After calibration</th>
<th>Before calibration</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSE</td>
<td>3.07</td>
<td>1992.717</td>
</tr>
<tr>
<td>( R^2 )</td>
<td>0.97</td>
<td>0.34</td>
</tr>
</tbody>
</table>

3.2. Model Verification

Following, some extreme conditions were applied to verify the model. To explain, the impact of zero rainfall on the rest of the variables affected from rainfall over the fifteen years, was evaluated. As Figure 7 shows, if rainfall reaches zero, the amount of aquifer and surface water inflow decreases (Figures 7.a and 7.c). In this condition, in order to meet all of the demands, the amount of depletion would rise; therefore, the amount of aquifer outflow would increase (Figure 7.b). Consequently, the potentiometric surface will be less than the actual value in each year (Figure 7.d).
In the next extreme condition test, the water allocation from Taleghan dam was assumed to be zero and then its impacts were evaluated (Figure 8). If water allocation from Taleghan dam is assumed as zero, the amount of the aquifer inflow is decreased because nearly 10% of this allocated water is used to recharge the aquifer as well as surface inflow which is supplied by Taleghan dam (Figures 8.a and 8.c). In addition, since some of the irrigation demand is provided by Taleghan dam, this assumption would require more extracting from the aquifer and put the groundwater level lower than the actual amount (Figures 8.b and 8.d).
3.3. Scenarios Analysis

After the calibration and the verification of the developed model, eighteen scenarios that were mentioned in table 1 were applied separately to the model. Table 3 shows farmers’ income, groundwater drawdown and normalized variables for all scenarios.

As mentioned, scenario 0 is the historical condition, where the aquifer was extremely depleted over the 15 years and according to table 3, farmers earned the maximum income of 2.49 billion dollars, therefore in this scenario, the FID was zero. Other scenarios have been evaluated based on scenario 0. Scenario 1, with a cropping pattern with no wheat, which saves groundwater with no serious impact on farmers’ income. This scenario has been led to reduce the rate of the negative balance, such that the amount of drawdown would be 1.3 meters, while farmers' income only is reduced by 8.6%. Due to the wheat guaranteed purchase program of the Iranian government [12], nearly 30% of farms are cultivated by wheat despite the marginal benefit of wheat crops in this area.

In the second scenario, removing barley would cause about 214 million dollars income deficiency, but improve drawdown by as much as 6.3 meters. It shows that the farmer’s income isn’t noticeably affected by the removal of barley, while it can make a credible positive impact on the groundwater level. In scenarios 2, 8, 9, 10, and 11 respectively, when barley, rapeseed, sugar beets, onion, and cotton were omitted, the farmers’ income diminished by less than 0.6 %, but the amount of avoided drawdown varied from 1% to 30%. Removing barley and sugar beet crops would avoid depletion by as much as 6.7 and 3.7 meters respectively. Therefore barely has a more potential to increase drawdown when compared to the other four crops in these scenarios.

In Scenarios 3, 5, 6, 7, 12, and 17, the impact of leaving out potato, maize, silage, cucumber, rapeseed, and pistachio crops respectively, was a decrease in farmers’ income from 1% to 5% and the avoided drawdown would differ from 0.48 to 4.7 meters. In these scenarios, scenario 17 could have the most impact to decrease farmers' income by about 100 million dollars, and prevent aquifer drawdown by about 0.78 meters. Scenarios 5, 6, and 12 would reduce farmers' income approximately 5.5 %, while these prevent the groundwater level from going down further; 4, 4.7 and 2.26 meters respectively.

<table>
<thead>
<tr>
<th>No.</th>
<th>Drawdown (m)</th>
<th>Avoiding drawdown (m)</th>
<th>ADW (%)</th>
<th>Total farmer’s net income (billion dollars)</th>
<th>Farmer’s net income deficiency (million dollars)</th>
<th>FID (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 0</td>
<td>20.80</td>
<td>0.00</td>
<td>0.00</td>
<td>2.49</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Scenario 1</td>
<td>1.38</td>
<td>19.42</td>
<td>93.35</td>
<td>2.28</td>
<td>214.84</td>
<td>8.61</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>14.54</td>
<td>6.27</td>
<td>30.12</td>
<td>2.49</td>
<td>3.76</td>
<td>0.15</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>19.43</td>
<td>1.17</td>
<td>5.63</td>
<td>2.46</td>
<td>33.96</td>
<td>1.36</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>16.89</td>
<td>3.91</td>
<td>18.80</td>
<td>2.21</td>
<td>281.50</td>
<td>11.29</td>
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<tr>
<td>Scenario 5</td>
<td>16.80</td>
<td>4.00</td>
<td>19.22</td>
<td>2.39</td>
<td>103.15</td>
<td>4.14</td>
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<tr>
<td>Scenario 6</td>
<td>16.02</td>
<td>4.78</td>
<td>22.98</td>
<td>2.36</td>
<td>134.32</td>
<td>5.38</td>
</tr>
<tr>
<td>Scenario 7</td>
<td>20.32</td>
<td>0.49</td>
<td>2.34</td>
<td>2.46</td>
<td>29.91</td>
<td>1.20</td>
</tr>
<tr>
<td>Scenario 8</td>
<td>20.34</td>
<td>0.46</td>
<td>2.22</td>
<td>2.49</td>
<td>4.04</td>
<td>0.16</td>
</tr>
<tr>
<td>Scenario 9</td>
<td>17.06</td>
<td>3.74</td>
<td>17.98</td>
<td>2.48</td>
<td>13.43</td>
<td>0.54</td>
</tr>
<tr>
<td>Scenario 10</td>
<td>20.56</td>
<td>0.24</td>
<td>1.16</td>
<td>2.49</td>
<td>3.81</td>
<td>0.15</td>
</tr>
<tr>
<td>Scenario 11</td>
<td>20.13</td>
<td>0.68</td>
<td>3.25</td>
<td>2.49</td>
<td>8.31</td>
<td>0.33</td>
</tr>
<tr>
<td>Scenario 12</td>
<td>18.54</td>
<td>2.26</td>
<td>10.87</td>
<td>2.39</td>
<td>107.69</td>
<td>4.32</td>
</tr>
<tr>
<td>Scenario 13</td>
<td>13.58</td>
<td>7.22</td>
<td>34.72</td>
<td>2.24</td>
<td>249.41</td>
<td>10.00</td>
</tr>
<tr>
<td>Scenario 14</td>
<td>18.75</td>
<td>2.06</td>
<td>9.89</td>
<td>2.35</td>
<td>144.09</td>
<td>5.78</td>
</tr>
<tr>
<td>Scenario 15</td>
<td>8.01</td>
<td>12.79</td>
<td>61.50</td>
<td>1.60</td>
<td>890.24</td>
<td>35.69</td>
</tr>
<tr>
<td>Scenario 16</td>
<td>18.24</td>
<td>2.56</td>
<td>12.30</td>
<td>2.32</td>
<td>171.63</td>
<td>6.88</td>
</tr>
<tr>
<td>Scenario 17</td>
<td>20.02</td>
<td>0.78</td>
<td>3.77</td>
<td>2.39</td>
<td>100.21</td>
<td>4.02</td>
</tr>
</tbody>
</table>

Removing watermelon and melon, peach and nectarine, and apple would reduce farmers’ income about 5-10 % in scenarios 13, 14, and 16 respectively, and these prevent groundwater depletion by about 34.72%, 9.89%, and 12.30 % respectively.
Scenario 15 decreases the farmers’ income dramatically by 36%. The scenario would help to reduce the historical drawdown to 8 meters. Thus, grapes play an important role for farmers’ income. Due to a good price, low cost, high export, and suitable climatic and soil conditions.

3.4. Indicators Analysis

The groundwater sustainability was assessed by SUI for each scenario. The index takes a positive or negative value, where high values (which are close to one) correspond to sustainable water use, and low values (especially negative ones) show groundwater abuse. The results infer that the SUI would be negative for all scenarios except scenario 1 (Fig.9). Meaning that, if the wheat crop was removed, then more sustainable conditions would be achieved.

Consequently, we would have unsustainable groundwater use under all scenarios. The government’s guaranteed price policy, has motiva...
4. Conclusion

The lack of fresh water is one of the primary world issues that has affected many countries in the arid and semi-arid area, such that human life is threatened in these regions. Groundwater depletion is one of the most common tragedies in these areas, causing many villages and cities to be abandoned. Inappropriate water resource management not only has led to groundwater depletion but also increased the spread of poverty. In this research, the effect of cropping patterns on the farmers’ income as well as the aquifer, on the Qazvin plain as a case study, with 1.3 yearly drawdowns, is studied using system dynamics.

The system dynamics that includes two subsystems, water volume, and farmers’ income were calibrated using a simple genetic algorithm that minimized the difference between the simulated data and the observed data over 15 years, from 1997 to 2011. By removing crops one by one, 18 scenarios were developed. In each scenario, the effect of the absence of each crop on the farmer’s income and the aquifer was assessed by SUI and WEI+ indicators.

The results show that wheat is the crop that has the most impact on the aquifer, such that cropping patterns with no wheat had the lowest drawdown and negligible impact on farmer’s income. Further, the results show that grapes play the most important role in the economical subsystem of the case study, such that removing them would reduce the farmers’ income by 36%.

Finally, increased wheat production has resulted from the government guaranteed purchase policy, even though it does not make as much money as other crops. This study revealed how the guaranteed purchase leads to groundwater depletion without any significant benefit for farmers.

5. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Wireless Video Monitoring of the Megacities Transport Infrastructure

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Received 19 February 2019; Accepted 08 May 2019

Abstract
The article discusses the development of megacities transport infrastructure. The problem of traffic congestion is touched upon, the task of optimal road design is justified. In the context of these issues gives a system of wireless video monitoring of traffic flows on base of digital panoramic video images. The main objective is to obtain a universal mathematical model for the description of a radio signal with any type of digital modulation. This will greatly facilitate the parametric calculation of the radio channel for image transmission and the design of the monitoring system. The objective is achieved by applying the Fourier method of separation of variables in combination with computer simulation methods. As result, a highly accurate universal mathematical model of radio signal with digital modulation is proposed. The scientific novelty of the model is that it allows to simulate the propagation of a radio signal with an arbitrary waveform. Thanks to this, the model covers almost all common types of digital modulation of the radio signal. In addition, the model takes into account the internal noise of the equipment and the external interference of the radio channel. The article describes in detail the process of solving the wave equation, underlying the model. Examples of modeling are given, the advantages and disadvantages of the model are indicated. Recommendations are made on its use for calculating radio channels and designing systems for analyzing and developing the transport infrastructure of a megacity.

Keywords: Metropolis; Transport Infrastructure; Development; Video Monitoring; Image Transmission; Wireless Communication; Radio Signal; Digital Modulation; Wave Equation; Fourier Method of Separation of Variables; Complex Time Basis.

1. Introduction

The problem of traffic congestion can be identified as one of the most pressing problems of megacities [1-5]. Traffic congestion is the main cause of the residents being late for work, they impede the work of the emergency services (fire brigade, first aid), and have a negative effect on the development of the city’s infrastructure. Today, this problem is being solved, mainly due to the commissioning of new lanes and “interchanges”. At the same time, as a rule, the road architecture develops "blindly", that is, without conducting a deep statistical analysis of the movement of traffic and the identification of patterns of their movement.

It is obvious that it is more profitable to design the transport infrastructure of the city purposefully (pre-analyze traffic at different times of the year, at different times of the day, on public holidays and on weekdays, peak times, and so on). Thus, the task of creating a system for monitoring traffic flows arises.

From a technical and economic point of view, aero video monitoring with using digital signal processing [6-12] and radio connection [13, 14] is the most promising direction. Quadcopters [15] can be used for panoramic aerial video images (Figure 1), video cameras on the roofs of high-rise buildings can also be installed, or special towers can be used.
Digital video images allow recognizing vehicle projections with a high degree of accuracy and automatically calculating traffic flow parameters such as the number of vehicles, their speeds, distances between them, and so on. However, to ensure high accuracy and reliability of the obtained statistical information, widescreen images of HD and Full HD formats are necessary, which require a high value of the carrier frequency of the radio signal for their wireless transmission.

In other words, to transmit more digital data in time, it is necessary to significantly increase the value of the carrier frequency of the radio signal (especially when it comes to conducting measurements in real time). The complexity of the design of the radio channel increases. Consequently, the problem of modeling and preliminary estimation of radio signal parameters with digital modulation (amplitude or frequency) arises at the design stage of the measuring system.

These signals, used for organizing digital communications, usually have a rather complicated form [16, 17], therefore, the existing mathematical apparatus does not fit to describe their transmission, and the development of new mathematical models is required. So, the necessity of the proposed technical solution is due to the absent of universal mathematical models of signal with arbitrary waveform. This greatly complicates the process of calculating the digital radio channel. This is especially true in the case of designing monitoring systems, based on small-sized aircraft, for which the size and power of the transmitter are most important.

Innovation of the technical solution is in mathematical model of propagation of arbitrary radio signal. It is a “linear” case of the wave equation, where it is assumed that the radio signal transmits along a straight line connecting the phase centers of the antennas of the source and receiver. The mathematical model is based on the partial differential wave equation, the solution of which was made possible by combination of Fourier method of separation of variables with computer simulation methods. The Fourier method allowed us to reduce the solution of the original partial differential equation to a set of ordinary differential equations. Computer methods have accelerated the process of finding a solution. The problem statement was given in the field of complex numbers.

2. Materials and Methods

The developed mathematical model was built on the basis of the well-known partial differential wave equation [18-20] with two terms with the second derivatives with respect to time and coordinate, solved by the Fourier method of separation of variables [20]. However, there are two beneficial differences in the proposed model from the standard wave equation and the standard method for its solution.

The first difference is that additionally introduces a term with the first derivative with respect to the coordinate, due to which the attenuation of the signal in space is taken into account. The second difference is the possibility of setting an arbitrary perturbing effect on the right-hand side, due to the transition to the mathematical apparatus of complex numbers (since only it allows to correctly combine even and odd harmonics of approximated functions in the Fourier method). Solution of the wave equation in partial derivatives (PDE) is reduced (Figure 2) to solving a set of ordinary differential equations (ODE).
For signals with digital modulation there is a certain ensemble of amplitudes in the case of amplitude modulation, and there is a certain ensemble of frequencies in the case of frequency modulation. The transition from one value of amplitude (frequency) to another value is carried out abruptly. The first kind of discontinuities are observed in the plots of amplitude and frequency changes in theory, which significantly complicates the mathematical model of the signal.

To take into account this feature of signals with digital modulation and to have the ability to specify an arbitrary disturbing action, a complex second-order partial differential equation with constant coefficients was chosen to describe the propagation of an arbitrary radio signal:

\[
q_1 \frac{\partial^2 u_1(t,x)}{\partial t^2} + q_2 \frac{\partial^2 u_2(t,x)}{\partial x^2} + q_3 \frac{\partial u_2(t,x)}{\partial x} = u_1(t,x)
\]

With boundary and initial conditions:

\[
\begin{align*}
  u_2(t,0) &= u_1(t,0) \quad \frac{\partial u_1(t,0)}{\partial t} = \frac{\partial u_2(t,0)}{\partial t} \\
  u_2(0,x) &= 0, \quad u_2(T,x) = 0
\end{align*}
\]

Where: \(u_1(t,x)\) is a complex function of input actions (equivalent of perturbation voltage), \(u_2(t,x)\) is the complex desired function (equivalent of propagation voltage), \(q_1, q_2, q_3\) are constant complex coefficients with zero imaginary parts for taking into account the properties of the medium, \(T\) is the period of decomposition of functions in time, \(t\) is time, \(x\) is the coordinate.

The solution of this equation is based on the Fourier method of separation of variables, similar to the real case, but using the time-based complex decomposition of the input actions function \(u_1(t,x)\) and the desired function \(u_2(t,x)\):

\[
\begin{align*}
  u_1(t,x) &= \sum_{k=0}^{K-1} c_{1,k}(x) e^{\frac{2\pi i k t}{T}}, \quad u_2(t,x) = \sum_{k=0}^{K-1} c_{2,k}(x) e^{\frac{2\pi i k t}{T}}
\end{align*}
\]

Where: \(c_{1,k}(x), c_{2,k}(x)\) are the complex coefficients of the expansion of the function of input actions and the desired function, respectively, \(k\) is the index of decomposition coefficients (index of harmonics), \(K\) is the quantity of coefficients (harmonics) of the decomposition, \(i\) - imaginary unit.

After obtaining expressions for partial derivatives based on the last formulas and their substitution in the original equation, according to the algorithm of actions in the Fourier method, to find a separate coefficient \(c_{2,k}(x)\), we have a complex differential equation:
\[ q_2 c_{2,1}(x) + q_3 c_{2,2}(x) - q_1 \left( \frac{2\pi k}{T} \right)^2 c_{2,1}(x) = c_{1,1}(x) \] (5)

In this case, the coefficients of decomposition of the input action \( c_{1,1}(x) \) are assumed to be already known (obtained from the preliminary column-by-column decomposition of the signal function \( u_1(t,x) \) during the simulation). The actual signal with digital modulation should be placed in the real part of \( u_1(t,x) \).

Thus, the original partial differential equation splits into some set of ordinary differential equations, and the search for its general solution reduces to finding all solutions of the equations of a given set, that is, finding all coefficients \( c_{2,1}(x) \) and then returning to the initial formula for the expansion functions \( u_2(t,x) \).

Each real ordinary differential equation for \( c_{2,k}(x) \) is solved by an approximate method using difference ratios instead of derivatives. Finally, the solution is extracted from the real part \( u_2(t,x) \), since the real part \( u_1(t,x) \) is used to set the signal.

We emphasize that the model works correctly only for the linear case, when the source equation includes functions from two arguments (in this case, the arguments are time \( t \) and \( x \) coordinate). The propagation delay between two points of space is not taken into account.

The efficiency of the mathematical model was confirmed by mathematical modeling, which was carried out using discrete programming methods and digital signal processing on a personal computer platform with Core 2 Duo processor and a clock frequency of 2 GHz system oscillator. The MATLAB environment of version R2009b, running under the Windows operating system, was used for writing and debugging software.

3. Results and Discussion

Results of modeling a «clear» radio signal are presented below (Figure 3). In the case of amplitude modulation, several levels of harmonic signal amplitude are used for transmitting digital symbols at the same frequency (information is transmitted by varying the amplitude of the signal by level). In the case of frequency modulation, on the contrary, several frequencies are used at a fixed signal amplitude (that is, information is transmitted due to the thickening and rarefaction of the harmonics).

The term equivalent of the propagation voltage is used for the voltage function on the graphs, since it is incorrect to speak of the “electromagnetic field voltage”. The equivalent of the propagation voltage is understood as the voltage value that would occur on the receiving antenna when it is placed “at a point” with a given \( x \) coordinate. In this case, the geometric dimensions of the antennas are neglected.

We also present the result of modeling a noisy radio signal with amplitude digital modulation as an example (Figure 4). This graph shows the effects of equipment noise and radio interference. The simulation was carried out in the MATLAB environment. The parameter of the propagation voltage equivalent, designated on the graphs as \( U \), in the mathematical model corresponds to the function \( u_2(t,x) \).
Figure 3. Equivalent of radio signal propagation voltage: (a) with amplitude digital modulation, (b) with frequency digital modulation

Figure 4. An example of a software implementation of a mathematical model for a signal with amplitude digital modulation (taking into account transmitter noise and external interference)

The radio signal in the model was transmitted at a distance of 100 m in the airspace. The amplitude of the voltage of the useful signal at the transmitting antenna was taken to be 1 V. The maximum noise level of the transmitting device was taken to be 10 μV, and the maximum voltage level of external interference was set to 100 μV. The carrier frequency used was 1 GHz. In this case, the total time interval for modeling was 30 ns. The figure below also shows the voltage in the receiver path (Figure 5).
According to the results of mathematical modeling, it is necessary to note the following important aspects regarding the proposed model. The intrinsic noise of the transmitting device at small values of their voltage level can be included in the model as an integral part of the full transmitted signal in the right-hand side. External interference with high voltage levels should be imposed after obtaining $u_2(t,x)$ and on the principle of superposition.

It should also be emphasized that the proposed mathematical model most reliably describes the propagation of a powerful radio signal over short distances. The greater the voltage amplitude of the transmitted useful signal, and the smaller the transmission distance, the more reliable the mathematical model works at a fixed coordinate point. This aspect is connected (Figure 6) with the issues of convergence of Fourier series (insufficiently good convergence leads to sharp bursts of the signal amplitude at the moments of its change, such bursts are usually called “outliers” in the literature on mathematical statistics). To eliminate this disadvantage, it is necessary to increase the number of Fourier coefficients.

We emphasize that the solution of the model should not be sought only by analytical methods, or only software (approximate methods). A joint approach is needed, since it may be necessary to solve several hundred ordinary differential equations to accurately reproduce the graph of the function sought (it is too laborious to do this “manually”).

In other words, the proposed mathematical model is the result of a combination of analytical (exact) and program (approximate) methods. It is based on the analytical Fourier method of separation of variables, but it is advisable to use computer simulation to find solutions of ordinary differential equations.

4. Conclusion

Based on the results of the research, first of all, we should highlight the following main advantages of the proposed mathematical model: the arbitrariness of the perturbing effect in the right side of the equation (which makes it possible to set a complex signal with digital modulation), the signal attenuation in space (which makes the model close to reality), as well as a convenient way to specify the properties of a particular medium or the boundary between several media (through the coefficients of the original equation).

The disadvantages of the proposed mathematical model include: significant computer time costs associated with the software solution of a set of ordinary differential equations (into which the original partial differential equation splits), partial loss of accuracy in describing functions with an insufficient number of Fourier expansion coefficients, and the theoretical complexity of the mathematical apparatus (complex basis of decomposition of functions).

In general, despite the indicated drawbacks, it should be noted that a promising complex mathematical model of radio signal propagation was developed. The model was obtained "at the junction" of analytical methods (Fourier method) and numerical methods for solving differential equations using difference schemes and computer simulation.
The theoretical foundations of the Fourier method of separation of variables and the mathematical apparatus of complex functions were widely used during the development.

As a result, the proposed model allowed setting an arbitrary useful signal of a complex shape, as well as taking into account external interference of the radio channel and attenuation of the signal in space depending on the properties of the specific environment, which opens up the possibility of modeling and preliminary assessment of the quality of receiving-transmitting devices with amplitude and frequency digital modulation of the signal. This is extremely important for design of measuring systems operating with widescreen video images.

The model is recommended to use for the selection of signal parameters with digital modulation and preliminary assessment of such important radio channel efficiency indicators as: required transmitter power (taking into account the working distance and signal attenuation index), radio channel capacity, signal-to-noise ratio at the receiver, calculated and mass-dimensional indicators of receiving and transmitting equipment.

Due to these advantages, the mathematical model can find the widest application in the design of wireless video monitoring systems of motor traffic flows to develop the transport infrastructure of the metropolis. In addition, the model can be used for wireless real-time traffic control, for example, by adjusting the switching time of traffic lights based on a statistical analysis of the transmitted video images.

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

6. References


Physiochemical Characterization and Dematerialization of Coal Class F Flyash Residues from Thermal Power Plant

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Received 08 February 2019; Accepted 18 April 2019

Abstract

Class F flyash has a low percentage content of lime and is considered as a leading category of flyash generated in India with an average utilization of nearly 55% of flyash produced by the coal-burning power plant. The coal Class F flyash residue sample has been collected from Harudaganj, Thermal Power Station India. The paper illustrates the outcome of the study carried out to examine all the relevant features of the chemical and physical properties of Class F flyash sample. Elementary quantitative results from point analysis, SEM/EDS, FTIR, and pH analysis have been done in the chemical analysis of the study. The physical characterization of the sample is done by several experimental approaches to compare all the relevant features of Class F flyash sample and common soil. The main objective of this study is to evaluate whether the locally available Class F flyash from Harduaganj Thermal Power Station India, will provide satisfactory performance in fully or partially replacement of common soil. The performance evaluation of flyash and soil in different test results included bulk density, specific gravity, plasticity, maximum dry density, optimum moisture content, and permeability in accordance with the relevant IS or ASTM standards. Finally, the reported research recommended the selection of Class F flyash sample with low-lime content that provided the close correlation of its physical properties to the common soil.

Keywords: Flyash; Class F; Class C; Utilization; Chemical; Physical; SEM/EDS.

1. Introduction

All over the world the coal based thermal power plant is one of the most effective resources for the production of electricity. A large variety of secondary materials are produced due to this process. Any material that are generated from coal-combustion processes are referred as a Coal-Combustion Product (CCP). Flyash is considered as one of the most widespread produced CCP among all possible types of CCPs that are generated at coal-burning power plants worldwide. The characterization of flyash can be discussed as the fine fraction of coal ash that exits in the combustion chamber of flue gas and is detained by air pollution control equipment at electric power plants [1]. From the shape point of view, the flyash are usually spherical and they solidify when in suspension form in exhaust gases. Flyash is generally composed of silica (SiO₂), alumina (Al₂O₃) and iron oxide (Fe₂O₃).

The chemical and physical characteristics of flyash vary typically on its anticipated utilization. Therefore, the particular needs for the use of flyash in soil stabilization or concrete are discussed in detail in references [2, 3]. According to the classification [2], flyash is classified into two classes’ viz. Class F and Class C. The content of silica, calcium,
iron and alumina in the ash is the major difference between these classes. The chemical characteristics of flyash samples are basically influenced by the chemical content of the coal burned (i.e., bituminous, anthracite, and lignite). The burning of older and harder anthracite and bituminous coal produces Class F flyash. The flyash so produced is pozzolanic in character and possess less than 10% lime, (CaO). The alumina and glassy silica of Class F flyash needs Portland cement, quicklime or hydrated lime as a cementing agent and in the presence of water it reacts and produces cementitious compounds. An alternate method is the addition of a chemical activator to Class F flyash like sodium silicate (water glass) that may lead to the formation of geopolymer. The combustion of younger sub-bituminous coal or lignite may generate flyash with pozzolanic properties and self-cementing properties. Class C flyash is known to harden in presence of water and gains strength over prolonged period. Class C flyash usually possess more than 20% lime (CaO). Class C with self-cementing property does not need an activator, unlike Class F. Alkali and sulfate (SO₄) amount are generally higher in Class C flyash. The aim of this investigation is to characterize the various chemical and physical properties of Class F flyash which discovers its potential utility. For this purpose; flyash is characterized with respect to its physical and chemical properties to look for the utilization of Class F flyash as resource material in construction industry.

1.1. Current Scenario of Flyash in India

In the world rank of coal and coal-based thermal power plants, India is the third largest producer which contributes to about 70% of the total installed capacity for generation of power [4]. Though, over 40% ash content produced by bituminous and sub-bituminous coal. There are 120 known coal-based thermal power plants existing in India which on an average generate 120-160 million tons of coal flyash [5]. The emerging amount of average generation and utilization data received during the last five years by Environmental Information Centre (ENVIS) on flyash is 166 million tons and 96 million tons [5]. The flyash generation and utilization in India during the year 2011-12, 2012-13, 2013-14, and 2014-15 and 2015-16 are shown in Table 1.

<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>Number of thermal power stations in India from which data was received</td>
<td>124</td>
<td>138</td>
<td>143</td>
<td>145</td>
<td>132</td>
</tr>
<tr>
<td>Installed capacity (MW)</td>
<td>1,05,925.3</td>
<td>1,20,312.30</td>
<td>1,33,381.30</td>
<td>1,38,915.80</td>
<td>1,30,428.80</td>
</tr>
<tr>
<td>Coal consumed (million tons)</td>
<td>437.41</td>
<td>482.97</td>
<td>523.52</td>
<td>549.72</td>
<td>251.69</td>
</tr>
<tr>
<td>Average ash content (%)</td>
<td>33.24</td>
<td>33.87</td>
<td>33.02</td>
<td>33.50</td>
<td>33.23</td>
</tr>
<tr>
<td>Flyash generation (million tons)</td>
<td>145.42</td>
<td>163.56</td>
<td>172.87</td>
<td>184.14</td>
<td>83.64</td>
</tr>
<tr>
<td>Flyash utilization (million tons)</td>
<td>85.05</td>
<td>100.37</td>
<td>99.62</td>
<td>102.54</td>
<td>46.87</td>
</tr>
<tr>
<td>Percentage utilization</td>
<td>58.48</td>
<td>61.37</td>
<td>57.63</td>
<td>55.69</td>
<td>56.04</td>
</tr>
</tbody>
</table>

Source: ENVIS Centre on Flyash Hosted by CSIR-Central Building Research Institute, Roorkee
Sponsored by Ministry of Environment, Forests & Climate Change, Govt. of India

Flyash generation and utilization data for the first half of 2015-16 (April, 2015 to Sept., 2015) has been received from 132 coal/lignite-based thermal power stations of different power utilities in India. The data received as on 15th March, 2016 has been investigated to obtain conclusions on the present known status of flyash generation and its utilization in India as a whole [6]. Flyash utilization percentage (of 146 thermal power stations) has increased during the first half of 2015-16 in comparison to the utilization during the first half of the previous year 2014-15[6]. ENVIS centre on flyash hosted by CSIR-Central Building Research Institute, Roorkee sponsored by Ministry of Environment, Forests and Climate Change, Government of India also shows the correlation in production and utilization of flyash from 1990 to expected 2030. The flyash generation and utilization scenario in India from 1990 to 2030 is represented in Figure 1.
1.2. Published Studies on Class F Flyash

The incorporation of the supplementary cementation materials like flyash and kaolin in the concrete mix may produce High Performance Concrete (HPC) [7]. The fibre reinforced flyash with lime stone dust brick (10FRFALSDB3') possessed the highest compressive strength of 9.155 MPa with 10% stone dust and 10% cement [8, 9]. After experimental tests on Class F flyash-based brick tiles, it is being concluded that tiles showed poor performance at lower compressive strength as compared to the conventional clay roof tiles. The study [10] shows that on increasing the percentage of Cement (C) at the fixed percentage of Treated Flyash (T.F.A) and Radish Stone Dust (R.S.D), the lower permeability values were found. Though permeability (k) falls sharply with the variation of Coarse Sand (C.S) with C, the value of k has been found in the range of $10^{-7}$ (closer to the value of clay available in the market for making bricks and roof tiles). Through new sets of experiment conducted by [11], its compressive strength increased to 30.65% as compared to the previous studies carried out by [9-11]. The study suggested that the flyash-scrap tire fiber composite offers a sustainable supplement to the traditional insulation which increases the efficiency of traditional insulation as well as help in reducing the percentage of disposed waste products [12].

The literature survey has established flyash as air and water pollution source. This resource may act as a material in construction industry, thereby approaching to a clean management of environment. Till a decade back, flyash was treated as waste material worldwide, but now it is developed as an environment saviour [13]. Various methods are suggested to use flyash, such as in the construction industry, agriculture, waste water treatment and management of environmental pollution [14]. The replacement of cement by flyash in concrete has resulted in the reduction of total voids, which may be attributed to the micro-filler effect of flyash. Consequently, there is a reduction of about 13.28% in permeability of pervious flyash-cement concrete [15]. The current annual worldwide production of coal ash is estimated about 700 million tons of which 70% is flyash at least and based on the more references, flyash can be used in different areas of building engineering thanks to its appropriate characteristics. It is obvious that the type of combusted coal has the significant impact on the chemical, physical and mineralogical characteristics of the flyash and its further utilization [16, 17].

The present study evaluates the application of class F flyash as a partial replacement of binder in concrete. The compressive strength of the fly ash samples showed low early compressive strength comparing to the control samples. However, due to pozzolanic reaction strength was improved gradually over a longer period of time, whereas control samples stopped the strength growth after 56 days of curing.

2. Materials and Methods

The flyash was obtained from Harduaganj, Thermal Power Station Aligarh, Uttar Pradesh India. The flyash was dried at 110°C in an electric oven for 2 h and stored in a desiccator for studying chemical and physical studies. For the comparison of physical properties, the sample of common soil was taken from the construction site of Sharda Mall Aligarh, India. Class F flyash sample was used after pretreatment with calcium hydroxide that enhances the cementitious properties of flyash sample. The sample’s surface morphology was analysed using JEOL JSM-6510LV Scanning Electron Microscope (SEM) assisted with Energy-Dispersive X-ray spectroscopy (EDX). EDS was used to characterize the samples in this study. The elemental analysis of the sample was done in “spot mode” in which the beam is localized
in a single area. The surface morphology of flyash particles were determined using SEM. For physical properties, Indian (IS) and (ASTM) Standard Test Methods were used. Figure 2 shows the picture of flyash and common soil sample used in the study.

![Fly Ash and Soil Sample](image)

**Figure 2. Typical Class F flyash and Soil Colour**

### 3. Results and Discussion

#### 3.1. Chemical Analysis of Class F Flyash

Class F flyash sample collected from the site and the common soil sample has been subjected to study herein. The physicochemical characteristics of Class F flyash are significant features for deciding economic utilization of flyash. It is relevant from the calculation that the percentage of CaO was found to be 1.31% in the given sample classifying it to be Class F flyash category defined by [2]. The compounds SiO$_2$ and Al$_2$O$_3$ are major constituents of flyash which were measured at their highest concentration i.e. 35.23% and 22.59% respectively. The constituents in flyash under study showed vast variations in their chemical composition due to the quality of Indian coal and lack of standardization in the plant machinery. The descending order of fixed carbon is pursued with increase in densities of the fractions which is due to higher percentage of mineral content of the fractions with smaller particle size.

The flyash pH value influence the time of setting of geopolymer paste [18], pH test was done by dissolving flyash into distilled water and then pH measurement was done after 24 h. Here in this study, pH value observed for flyash sample was 7.30 whereas pH value of the soil sample used as standard was 7.80. As per the literature review, flyash pH value ranges from 8 to 11 and it tends towards rapid setting. pH value of flyash has close correlation with its CaO content. High CaO content in flyash resulted in higher pH value [18]. Further, the sample of flyash was subjected to FTIR to study the presence of functional groups in the sample as shown in Figure 3. For the chemical characterization of flyash, FTIR technique is applied. FTIR data indicates the presence of functional groups on the surface of flyash samples. FTIR analysis represents band at 1041 cm$^{-1}$ showing Si-O-Si and Si-O structure. The band is observed at 795 cm$^{-1}$ representing Si-O structure, Si-O-Al structure (Al, Mg)-O-H Al-O-(Mg, Al) structure [19].

![FTIR of Class F flyash sample](image)

**Figure 3. FTIR of Class F flyash sample**
The flyash samples were also examined for their chemical composition. The identified elements in the ash sample were found to be C, O, Al, Si, K, Ca and Ti in various compound forms (Al₂O₃, SiO₂, K₂O, CaO, TiO₂ etc.). The quantitative result from point analysis of Class F flyash used in the study is shown in Table 2. The chemical composition may be cross verified by SEM-EDS technique as well that provide detailed information regarding morphology and surface texture of individual particles with elemental composition of samples. SEM is most widely used for the chemical characterization of ash.

Table 2. List of elements with their weight (%) and atomic (%) present in Class F flyash

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight %</th>
<th>Atomic %</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>14.42</td>
<td>21.05</td>
</tr>
<tr>
<td>O</td>
<td>54.59</td>
<td>59.83</td>
</tr>
<tr>
<td>Al</td>
<td>11.96</td>
<td>7.77</td>
</tr>
<tr>
<td>Si</td>
<td>16.47</td>
<td>10.29</td>
</tr>
<tr>
<td>K</td>
<td>0.73</td>
<td>0.33</td>
</tr>
<tr>
<td>Ca</td>
<td>0.94</td>
<td>0.41</td>
</tr>
<tr>
<td>Ti</td>
<td>0.88</td>
<td>0.32</td>
</tr>
</tbody>
</table>

3.2. SEM/EDS Analysis

The surface morphology characterization has a critical role in understanding the chemical and physical behavior of the material. SEM is a technique used to study different modes of association and detection of irregularities on surface. SEM is used in the study to investigate the surface morphology of the sample. Figure 4 shows SEM images recorded for sample Class F, flyash surface at x2000, x5000 and x6000 magnifications. Figure 4(a) and 4(b) shows the presence of irregular shaped particles of variable size, covered with relatively smooth grains of quartz. The micrographs in Figure 4(c) and 4(d) also designated dark areas as organic materials, light areas as mineral matter and gray as mixture of coal and ash. The solid and porous part indicated the presence of mineral matter most likely quartz, which is supported in earlier study [19]. The partially burnt coal particles were shown by irregular black porous parts. The particle size 10µm at WD 13mm appeared to be spherical with small bulging of siliceous and aluminous glass in Figure. 4(e). EDS study of flyash sample suggested the presence of Carbon, Oxygen, Aluminum, Silicon, Potassium, Calcium, and Titanium as the primary element. Thus, it is clear from the discussion given above that SEM/EDS is useful tool to study the morphology and surface texture of individual particles as well as elemental composition of samples. The identified elements in the flyash samples were found to be C, O, Al, Si, K, Ca and Ti, in various compounds (Al₂O₃, SiO₂, K₂O, CaO, TiO₂ etc.) as determined by EDS.

(a) SEM image of class F flyash sample (size: 2µm) at WD 13mm; (b) SEM image of class F flyash sample (size: 2µm) at WD 14mm
Figure 4. SEM micrograph of Class F flyash with different particles size a) 2µm at WD 13 mm; b) 2µm at WD 14 mm; c) 5µm at WD 13 mm; d) 5µm at WD 14 mm and e) 10µm at WD 13 mm

3.3. Physical Analysis of Class F Flyash

The physical analysis is one of the most important parameter for selection and consideration of material in the civil engineering construction industry. Its geotechnical property makes it a good substitute of soil and the required percentage provide the general range of physical geotechnical properties available in the flyash sample. As determined in the present study, the physical properties of Class F flyash are listed in Table 3. The physical properties of locally available common Indian soil are selected for the comparison as provided in Table 3. The typical variations of comparative study obtained by Table 3 and plotted in Figure 5. The sample of locally available common soil tested at 1.5m, 3.0m, 4.5m, 6.0m and 7.5m for comparison with Class F flyash sample is shown in Figure 5. It is found in the study that the physical properties of Class F flyash is very close to the relative values of common soil though it may differ from one country to another on their geographical conditions.

There are several reasons for the difference in soil regionally. The most significant factors include the parent soil, the climate and terrain of the region as well as the type of plant life and vegetation present and of course, human influence. In this study, the focus is on the geotechnical functions of the flyash and its comparison with the common soil which is an important criterion to replace any natural material. The strength and durability are two important factors to replace any material in the construction industry. A material is considered as a suitable building material if it possesses engineering properties suitable enough for construction works. These properties of building materials are responsible for its quality and capacity and help to decide their applications. A series of experiments conducted [20] had shown to improve the soil properties viz. texture, structure and bulk density. The permeability of clay loam soil increased from 0.54cm/hr to 2.14cm/hr by the addition of 50% flyash whereas it decreased from 23.80cm/hr to 9.67cm/hr in sandy soil by 50% fly-ash addition. The water holding capacity of sandy soil also increased from 0.38cm/cm to 0.53cm/cm at 50% level. Indian flyash is alkaline in nature; hence, its application for agricultural soil could increase the soil pH and thereby neutralize acidic soil [21].
Table 3. Summary of test results for different experiments on Class F flyash

<table>
<thead>
<tr>
<th>Experimental Parameters</th>
<th>Class F Flyash</th>
<th>Common soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Density (gm/cc)</td>
<td>1.25</td>
<td>1.3–1.7</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.12</td>
<td>2.5-2.8</td>
</tr>
<tr>
<td>Plasticity</td>
<td>Lower or non-plastic</td>
<td>Lower or non-plastic</td>
</tr>
<tr>
<td>Maximum Dry Density (gm/cc)</td>
<td>1.2875</td>
<td>1.3-2.4</td>
</tr>
<tr>
<td>Optimum Moisture Content (%)</td>
<td>18</td>
<td>5.0-30.0</td>
</tr>
<tr>
<td>Angle of Internal Friction(degrees)</td>
<td>28</td>
<td>30-40</td>
</tr>
<tr>
<td>Cohesion (kN/m²)</td>
<td>2.1</td>
<td>Negligible</td>
</tr>
<tr>
<td>Permeability (m/sec)</td>
<td>$1.650 \times 10^{-5}$</td>
<td>$8 \times 10^{-5} - 7 \times 10^{-4}$</td>
</tr>
<tr>
<td>Shrinkage Limit (Vol stability)</td>
<td>Higher</td>
<td>Low - high</td>
</tr>
<tr>
<td>Grain size</td>
<td>Major fine sand range / and very small percent of clay size particles</td>
<td>Major sand size fraction / silt and clay fraction and small percent of gravel size fraction</td>
</tr>
<tr>
<td>Clay (percent)</td>
<td>Negligible</td>
<td>Low - medium</td>
</tr>
<tr>
<td>Free Swell Index</td>
<td>Very low</td>
<td>Very low</td>
</tr>
<tr>
<td>Classification (Texture)</td>
<td>Sandy silt to silty loam</td>
<td>Sandy clay, silty clay, clay loam and silt loam</td>
</tr>
<tr>
<td>Water Holding Capacity (WHC) (percent)</td>
<td>40-60</td>
<td>10-70</td>
</tr>
<tr>
<td>Porosity (percent)</td>
<td>30-65</td>
<td>15-75</td>
</tr>
</tbody>
</table>

Figure 5. Comparative study of Class F sample and common soil sample

The specific gravity of flyash is 2.12 as determined by density bottle method. The sample of flyash and common soils were analyzed by using different sieve sizes. For consistency, the gradation of flyash was kept same as that of the common soils. The gradation of Class F sample and common soils were obtained by sieving as shown in Table 4. In the case of flyash sample, % finer was found to be 99% by hydrometer analysis. It was found that the diameter of the flyash particles ranges from 0.005 to 0.600 mm as shown in Figure 6. The locally available common soil sieve analysis results are plotted in Figure 7 that shows about 65% particles passing 0.075 mm fall in clay and silt range. By the above discussion, it has been figured out that the Class F flyash particles are coarser than common soil.
Table 4. Gradation of Class F flyash and common soil samples

<table>
<thead>
<tr>
<th>Opening size (mm)</th>
<th>Class F Sample</th>
<th>Percent Passing (CS-1.5)</th>
<th>Percent Passing (CS-3)</th>
<th>Percent Passing (CS-4.5)</th>
<th>Percent Passing (CS-6)</th>
<th>Percent Passing (CS-7.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.425</td>
<td>99.0</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>0.300</td>
<td>98.4</td>
<td>92.0</td>
<td>99.8</td>
<td>98.0</td>
<td>97.6</td>
<td>100</td>
</tr>
<tr>
<td>0.212</td>
<td>93.4</td>
<td>85.5</td>
<td>96.5</td>
<td>94.0</td>
<td>93.8</td>
<td>99.4</td>
</tr>
<tr>
<td>0.150</td>
<td>82.6</td>
<td>78.0</td>
<td>89.2</td>
<td>89.0</td>
<td>87.2</td>
<td>96.0</td>
</tr>
<tr>
<td>0.075</td>
<td>12.6</td>
<td>61.0</td>
<td>57.3</td>
<td>77.0</td>
<td>58.7</td>
<td>69.6</td>
</tr>
</tbody>
</table>

Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) \( \gamma_d \) of the Class F flyash and common soil were determined by the Proctor’s compaction test. The main aim of this test is to arrive at a standard which may serve as a guide and basis of comparison for the field compaction. Finally, a graph of moisture content vs dry density of Class F sample is plotted as shown in Figure 7. Locally available soil at 1.5m, 3.0m, 4.5m, 6.0m and 7.5m is shown in Figure 8. OMC is found to be 18% against \( \gamma_d 1.2875 \) g/cc for Class F sample which is close to the range of common soil found maximum at 6.0 m depth from the ground and 13.68 % against the \( \gamma_d 2.15 \) g/cc.
As seen in Table 3, the permeability of the Class F sample was found $1.650 \times 10^{-5}$ m/sec. In general, the permeability of soil is measured using a permeameter test following either the constant-head or falling-head method. The former method is recommended for coarse-grained soils where $k$ is expected to be smaller than $10^{-5}$ cm/sec, or when the soil contains 90% or more particles that are retained on 75µm sieve [22]. Conversely, the falling-head test is suited for testing fine-grained soils where $k$ value is expected to be within the range of $10^{-5}$ to $10^{-8}$ cm/sec, or when the soil contains 10% or more particles passing 75µm sieve. Therefore, the falling-head method was selected in this study for testing Class F flyash sample containing fine particles such as silica.

4. Control Point of the Study

The tonnes of flyash produced every year because of the massive coal consumption have proved to be hindrance in developing optimized and cost effective techniques for reusability. The recycling of flyash will conserve the natural raw materials and reduce the disposal cost. It will also create new revenues and business opportunities while protecting the environment. Based on the experimental results, the following control point can be drawn:

- The identified elements in the flyash sample were found to be C, O, Al, Si, K, Ca and Ti in various compound forms ($\text{Al}_2\text{O}_3$, $\text{SiO}_2$, $\text{K}_2\text{O}$, $\text{CaO}$, $\text{TiO}_2$ etc.).
- The percentage of CaO was found to be 1.31% in the given sample classifying it to be Class F flyash category.
- pH value observed for flyash sample was 7.30 whereas pH value of the soil sample used as standard was 7.80. As per the literature review, flyash pH value ranges from 8 to 11 and it tends towards rapid setting.
- pH value of flyash has a close interconnection with its CaO content. Excessive CaO content in flyash upshot in higher pH value.
- FTIR analysis represents the band at 1041 cm$^{-1}$ showing Si-O-Si and Si-O structure. The band is observed at 795 cm$^{-1}$ representing Si-O structure, Si-O-Al structure.
- The specific gravity and bulk density of Class F sample were found in the range of common soil sample i.e., is 2.12 and 1.25.
- The sample of flyash and common soils were analyzed by using different sieve sizes. The particles of Class F sample found coarser than a common sample.
- O.M.C is found to be 18% against MDD ($\gamma_d$) 1.2875 g/cc for Class F sample which is close to the range of common soil found maximum at 6.0 m depth from the ground and 13.68 % against the ($\gamma_d$) 2.15 g/cc.
- The porosity test results of Class F sample show higher initial value than common soil but variation is within the acceptable limit.
- The significance of moisture composition is very high in the permeability of fresh Class F sample. Such computation and evaluation can be utilized for stability analysis of earthen structures. The permeability results of the present study have been found in the perfect match with the range of common soil.
5. Conclusion

In India, the majority of flyash produced fall in Class F. ASTM C618 Class F flyash samples have been examined in the present study. This study shows the chemical and physical suitability of Class F flyash in the construction industry. Since flyash ties up free lime which leads to less bleed voids, it leads to a considerable reduction of permeability to water and sulfate as aggressive chemical. Moreover, in case of sulfate attacks the experimental results in published studies on Class F flyash shows that use of 20% flyash as replacement of Portland cement cause a slight difference in strength properties of the samples. It also reveals that all physical properties are much closer than available common soil. By using Class F flyash, the amount of soil used in the production of soil based construction materials i.e., bricks, roof tiles, and blocks etc. can be reduced. The economic benefit can be achieved by using flyash as a pozzolanic addition in the concrete mixture and mixed mortar. In conclusion, the use of additional waste materials provide both durable and economic building construction and ecological balance. For the Indian condition, it is recommended that Class F flyash can be used as a general fill in construction activities i.e., buildings, roads, embankment, and low lying areas. In contrast with generally used fill material (local soils) Class F flyash is a lightweight material. India is an agriculture-based country. The excessive and unsuitable usage of soil result in nutrient exhaustion, abrasion and other forms of losses, the soil productivity declines; it lessens the accessible areal domain for agricultural utility. By utilizing flyash as fill material, an equal volume of topsoil which will otherwise be used in filling can be saved.

6. Acknowledgement

The authors acknowledge their gratitude to USIF, A.M.U Aligarh, India for providing SEM/EDX data. In addition, authors are also thankful to soil mechanics laboratory staff especially Mr. Hamid A.M.U Aligarh, India, for extending the help in experimental work.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


[5] The Built Envison; ENVIS; Newsletter from CBRI ENVIS Centre of fly ash; Central Building Research Institute Roorkee; Roorkee, 2016.


A Comparative Study on the Flexural Behaviour of Rubberized and Hybrid Rubberized Reinforced Concrete Beams

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Received 06 February 2019; Accepted 17 April 2019

Abstract

This paper aims to investigate the flexural behaviour of the rubberized and hybrid rubberized reinforced concrete beams. A total of fourteen beams, 150×200 mm in cross-section with 1000 mm in length, were subject to a laboratory test over an effective span of 900 mm. The sand river aggregate was replaced by 10%, 12.5%, and 15% of crumb rubber (volume). The hybrid structure contained two double layers: 1) rubberized reinforcement concrete at the top layer of the beam and 2) reinforcement concrete at the bottom layer of the concrete beam. The static responses by the flexural test of all the beams were evaluated in terms of their fresh properties, failure patterns, total energy, flexural strength, stiffness, and ultimate deflection, modulus of rupture, strain capacity, and ductility index. The results showed that there were improvements when the hybrid beams were used in most cases such as failure pattern, ultimate load, stiffness, modulus of rupture, and stress. The rubberized concrete beams showed improvements in the strain capacity as illustrated in strain gauges and stress-strain curves, toughness, ultimate deflection, and ductility index. The findings of the study revealed an improved performance with the use of the hybrid beams. This has resulted in the implementation of innovative civil engineering applications in the engineering sustainable structures.

Keywords: Rubberized Concrete; Hybrid-Rubberized Concrete; Crumb Rubber; Double Layers.

1. Introduction

There exist billions of scrap car tires per annum, which represents a significant environmental issue around the globe [1]. Significant benefits can, therefore, be achieved when these expired tires are reused in different civil engineering applications such as concrete. Sustainable concrete can be obtained and the dangerous substances of tires in the environment can be reduced [2]. Regarding such a critical environmental issue, studies (small-scale specimens) were published during the last twenty years. These studies focused on the use of crumb rubber particles as an alternative to using sand aggregates replacement at different ratios. The findings showed a decrease in the mechanical properties of concrete and low workability, which were increased when the percentage of rubber replacement increased [3]. However, different findings revealed that toughness, strain capacity, ductility, and cracking resistance and reduction of self-weight have improved [4, 5]. Ismail et al. [6] Conducted flexural testing in large-scale of structural beams made of rubberized concrete ranged from 5 to 15% of fine aggregate replacement. They pointed out that, with the increase of crumb rubber it will lead to the reduction of crack widths and increased the number, reduce self-weight of concrete, decrease in toughness when exceeded 15% of crumb rubber contents. Another study on large scale beams was carried out by Mendis et al. [7] to evaluate the efficacy of different ratio of crumbled rubber on beams with and without shear reinforcements.

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It was found that the rubber contents influenced the shear capacity, yet, the relationship between the rubber ratio and shear capacity were not sufficient for the determination. Therefore, the confidence level of using crumb rubber still needs to be build up to be effectively used in structural members with a new arrangement and design of material distribution in the concrete mix.

This means that the hybrid concrete structures can be used to remove some obstacles because of using the crumb rubber in conventional concrete. This can be useful for improving the confidence level. Mohankar et al. [8] defined the term ‘hybrid’ as a composite of two or several types of materials with different shapes or dimensions. Commercial products that combine various types of materials (such as steel and plastic) in concrete composites are widely available [9]. The hybrid structure in the construction industry is a combination of two layers containing the precast and in-situ concrete. This type of structure has special conditions and it undergoes a specific process for construction based on the client’s requirements [10]. In the context of this study, the hybrid concrete structure is significant because it is a combination of two or more different materials within the concrete, which can improve the hardened properties [11].

Promising results have been reported in previous studies that investigated the hybrid structure on a small scale including the rubberized concrete. Li et al. [12] investigated the isolated hybrid structure, which consists of two layers: the rubberized concrete was placed at the bottom of the beams up to 30% of their overall depth and the rest was filled with normal concrete. The findings revealed that these types of structure were useful in reducing the structure response when exposed to load. In another study, Al-Tayeb et al. [13] investigated the rubberized concrete and the hybrid structure by using a prism adding 5%, 10%, and 20% of rubber. The hybrid structure consists of two layers: the rubberized concrete layer and the normal concrete layer. The rubberized concrete was placed on the top of the layer and was filled up to the height of 50%. The normal concrete was put at the bottom of the beams. The findings revealed better performance in dynamic loading than in static loading.

Norman [14] investigated small-size beams of rubberized and hybrid-rubberized concrete under static and dynamic load. The main variables in the experiment were crumb rubber with different percentages (5%, 10%, 15%, 17.5%, 20%, 22.5%, and 25%) of partial replacement of fine aggregate located at the top of 50% in height for the hybrid structure. The plain concrete was placed at the bottom. The results showed that the hybrid structure with a rubberized layer at the top absorbed high flexural energy in an impact load than in a static load. Previous studies tested the hybrid members that include the rubberized concrete. For example, Abqari [15] carried out a comparative study including the rubberized concrete and hybrid-rubberized concrete beam with two layers. The top was given for conventional concrete, while the bottom was given for rubberized concrete at 20% of the volume for fine aggregate replacement. Their height of filled are based upon the neutral axis. The results revealed that the performance was the same for the rubberized reinforced concrete beam and the hybrid reinforced concrete beam during the first crack loading. However, the results revealed that the stiffness of the hybrid structure exhibited better performance than the rubberized concrete. This can be attributed to the hybrid reinforced concrete beam deflection, which is slightly lower than the rubberized reinforced concrete beam.

Upon reviewing the previously conducted studies in this section, it can be concluded that there is a gap in the literature. This gap is related to the flexural behaviour of the rubberized and hybrid rubberized reinforced concrete beams at a large scale. Therefore, the main purpose of this study is to examine the flexural behaviour of a novel type of hybrid reinforced concrete beams with the top layer that consists of rubberized concrete at 10%, 12.5%, and 15% by volume of fine aggregate. This paper contributes to sustainable concrete structures by providing a potential concrete structure application, which is used in various practical works.

2. Experimental Program

2.1. Materials

One of the materials that were used in the experimental works is type 1 of ordinary Portland cement with a relative density of 3.15. The natural crushed aggregate of 10 mm maximum size and natural river sand were used as coarse aggregates and fine aggregates, respectively. The relative density, fineness modulus and absorption of fine aggregates were 2.64%, 3.40%, and 1%, respectively. A crumb rubber aggregates with a maximum size up to 4.75 mm, with a specific gravity of 1.22. The gradation curves for coarse, fine and rubber aggregate was obtained by sieve analysis as illustrated in Figure1. Superplasticizer (SP) was used with a specific gravity of 1.06 to adjust the workability of the mixture.
2.2. Concrete Mixtures

Seven mixtures were prepared, and each mix consisted of three cylinders of 100 mm in diameter and 200 mm in height, three cubes of 100 mm size, two beams of 1000 ×150×200 mm dimensions that were reinforced with four bars of 10 mm longitudinal reinforcement and stirrups of 6 mm in diameter for shear at 125 mm spacing between center to center as shown in Figure 2. All the steel bars had an average yield stress of 428 MPa. The controlled specimens were prepared without any inclusion of crumb rubber, while three mixtures of rubberized concrete were prepared with the inclusion of crumb rubber ratio at 10%, 12.5%, and 15% of fine aggregate replacement. The hybrid structure was cast into two equal layers. The first layer was for normal concrete, whereas the second layer was for rubberized concrete and it consists of three mixture types based on the percentage of crumb rubber which varied from 10% to 15%. The details of the rubberized and hybrid rubberized concrete beams, cubes, cylinder, and its mix proportions are illustrated in Figure 3 and Table 1.
Figure 3. Details of Rubberized and Hybrid Rubberized Concrete

Table 1. Concrete mix designs a) Controlled and rubberized concrete, b) Hybrid-rubberized concrete

(a)

<table>
<thead>
<tr>
<th>Type of Mix</th>
<th>Name</th>
<th>Cement (kg/m³)</th>
<th>Coarse Aggregates (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Fine Aggregates (kg/m³)</th>
<th>Crumb rubber (kg/m³)</th>
<th>Superplasticizer (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Controlled</td>
<td>RC</td>
<td>360</td>
<td>1060</td>
<td>173</td>
<td>713</td>
<td>0</td>
<td>0</td>
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<tr>
<td>Rubberized concrete</td>
<td>RCB1</td>
<td>360</td>
<td>1060</td>
<td>173</td>
<td>641.7</td>
<td>10 %</td>
<td>32.86</td>
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<tr>
<td></td>
<td>RCB2</td>
<td>360</td>
<td>1060</td>
<td>173</td>
<td>623.88</td>
<td>12.5%</td>
<td>41.08</td>
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<tr>
<td></td>
<td>RCB3</td>
<td>360</td>
<td>1060</td>
<td>173</td>
<td>606.05</td>
<td>15%</td>
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</tbody>
</table>

(b)

<table>
<thead>
<tr>
<th>Type of Mix</th>
<th>Name</th>
<th>Cement (kg/m³)</th>
<th>Coarse Aggregates (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Fine Aggregates (kg/m³) First layer</th>
<th>Second layer</th>
<th>Crumb rubber (kg/m³) First layer</th>
<th>Second layer</th>
<th>Superplasticizer (kg/m³) First layer</th>
<th>Second layer</th>
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<tr>
<td>Hybrid</td>
<td>HRCB4</td>
<td>360</td>
<td>1060</td>
<td>173</td>
<td>641.7</td>
<td>713</td>
<td>10.00%</td>
<td>32.86</td>
<td>1.08</td>
<td>0.9</td>
</tr>
<tr>
<td>Rubberized concrete</td>
<td>HRCB5</td>
<td>360</td>
<td>1060</td>
<td>173</td>
<td>623.88</td>
<td>713</td>
<td>12.50%</td>
<td>41.08</td>
<td>1.08</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>HRCB6</td>
<td>360</td>
<td>1060</td>
<td>173</td>
<td>606.05</td>
<td>713</td>
<td>15.00%</td>
<td>49.38</td>
<td>1.08</td>
<td>0.9</td>
</tr>
</tbody>
</table>
3. Testing

3.1. Mechanical Properties Testing

A total of 63 specimens by 21 and 42 for cube and cylinders, respectively were cast according to ASTM C192 [16] for each mixture. In the hybrid (layered), cube and cylinder were cast with the same mixture at the time of casting the reinforcement beams for each layered as shown in Figure 4. The compressive strength, split test, density, and modulus elasticity were determined after curing was complete for 28 days in conditions that were like those of the tested beams. Figure 5 shows the research methodology of this study for materials properties and reinforcement beams.

![Figure 4. Cast and Test of Hybrid Structure for Standard Specimens](image)

![Figure 5. Research methodology flowchart](image)
3.2. Flexure Test Setup, Instrumentation, and Loading Procedure

The beams were tested under four points load to determine its flexural strength. The linear variable differential transformer (LVDT) was used to record and measure the mid-span deflection until failure. A strain gauge was mounted at the main bottom steel bar and at the top middle of the concrete surface with different lengths to measure the strain during the loading. This strain gauge was manufactured by electronic instrument-Japan. In this strain gauge, 30 mm length used for concrete, while 5 mm length for longitudinal reinforcement. An incremental load was applied to the tested beams, while the first crack was monitored and observed by the naked eye at different levels of loadings (first cracking load and ultimate load). These were detected, marked, photted, and recorded. The details of the testing setup are shown in Figure 6.

![Figure 6. Beam Set-up a) Schematic Beams Test, b) Real Beams Test](image)

4. Results and Discussion

4.1. Mechanical Properties

Table 2 illustrates the three specimens’ average of the mechanical properties (compressive strength, split tensile strength, and static modulus of elasticity) of all the mixtures under a similar condition like the structural arrangement to that of the tested beams. In general, the reductions accrued in the overall results of the mechanical properties at the age of 28 days with increasing the crumb rubber ratios from 10% to 15%. By examining mixture RC (the controlled one) to RCB3 (15Cr), the compressive strength for cubes dropped from 8.2 % to 16.5% at crumb rubber percentage, which varies from 10% to 15% by replacement of sand, respectively. Several studies have previously reported the decline in
compressive strength subjected to rubber contents by revealing the negative effect [6, 14, 17, 18]. The nature of the crumb hydrophobic allowed to increase the voids by air in the concrete mixtures and lead to concentrate the stress across micro cracks that caused by rubber thereby reduction in hardness strength [19]. The reductions of compressive strength at the same ratios of crumb rubber contents were considerably higher in cylinder, which ranged from 24.1% to 40%. The huge reductions can be mainly attributed to the cylindrical shape and ratio of the height to diameter as indicated by Yi et al. [20].

Table 2. Results of the mechanical properties

<table>
<thead>
<tr>
<th>Types of mix</th>
<th>Compressive strength, $f_c$ (MPa)</th>
<th>Density (kg/m³)</th>
<th>Static modulus of elasticity (GPa)</th>
<th>Split tensile strength, $fsp$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC (Controlled)</td>
<td>48.5 41.30</td>
<td>2370.12</td>
<td>31.9</td>
<td>3.6</td>
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<tr>
<td>RCB1</td>
<td>44.5 31.35</td>
<td>2292.62</td>
<td>26.4</td>
<td>2.9</td>
</tr>
<tr>
<td>RCB2</td>
<td>42.9 26.68</td>
<td>2269.01</td>
<td>24.0</td>
<td>2.8</td>
</tr>
<tr>
<td>RCB3</td>
<td>40.5 24.75</td>
<td>2259.12</td>
<td>23.0</td>
<td>2.7</td>
</tr>
<tr>
<td>HRCB4</td>
<td>46.4 34.11</td>
<td>2323.93</td>
<td>28.1</td>
<td>3.4</td>
</tr>
<tr>
<td>HRCB5</td>
<td>44.5 28.56</td>
<td>2300.33</td>
<td>25.5</td>
<td>3.3</td>
</tr>
<tr>
<td>HRCB6</td>
<td>42.9 26.41</td>
<td>2289.71</td>
<td>24.2</td>
<td>2.9</td>
</tr>
</tbody>
</table>

The hybrid cubical and cylindrical structure contains rubberized concrete at the top and normal concrete at the bottom. There was a lower decrement (improvement) in compression strength compared to non-hybrid (RC, RCB1, RCB2, and RCB3). It was varied in cubes from 2.9 to 11.6. However, in the cylindrical structure, it ranged from 17.4% to 36.1% at 10 to 15% of crumb rubber volume, respectively. This is because the rubber contents to half of the volume specimens compared fully specimen’s rubberized concrete. Similar effects of crumb rubber were obtained in density as illustrated in Table 2 for the rubberized and hybrid rubberized concrete in each mixture. These results were compared with previous studies [6, 18, 21-22] that were conducted on the rubberized concrete only. One possible justification is that the relative density of sand in this study was doubled at 2.65 in contrast with the relative density of crumb rubber at 1.22. Zheng et al. [23] supported this reason and argued that the lower specific gravity of crumb rubber can be attributed to this reduction with various outcomes based on the practical size of rubber. Also, Richardson et al [24] reported that 1.5% of fine aggregate replacement by crumb rubber weight lead to increasing in the air content by 26%, consequently decreasing the density by 2%. In contrast, the hybrid cubical and cylindrical structure showed an increase in the overall unit density than the rubberized concrete. The modulus of elasticity can be calculated directly based on the ACI formula [25] as shown below:

$$E = Wc^{1.5}X0.043\sqrt{fc}$$

Where $E$ = The modulus of elasticity (GPa), $Wc$ = cube density (kg/m³), $fc$ = Compressive strength of cylinder (MPa).

Ahmed [14] and Zheng [23] used this expression to predict the modulus of elasticity based on the cube’s density (Kg/m³) and compressive cylinder strength (MPa) values, which were obtained based on their experiment. According to Table 2, there are strong relationships between the compressive strength and the modulus of elasticity in reductions that are subject to crumb rubber contents. It is also expected that these reductions exist because the fine aggregate had a higher modulus of elasticity in contrast with the crumb rubber aggregate [14]. In the hybrid structure, the reduction of a modulus of elasticity has improved from 4% to 5% approximately compared with the fully rubberized concrete at 10%, to 15% of crumb rubber ratios.

The splitting tensile results were followed by the reductions of compressive strength and the modulus of elasticity that were caused by incorporation of the crumb rubber as a partial replacement of fine aggregates. The value results of controlled splitting strength were 3.60 MPa. After that, it decremented to 3.16, 2.83, and 2.73 MPa when the fine aggregate was partially replaced by the crumb rubber at 10, 12.5, and 15%. This finding was found to be inconsistent with previous studies’ findings [6, 14, 20, 21]. In the cylindrical hybrid structure, there was an improvement in splitting tensile by 8.2%, 16%, and 5%, respectively when compared with the rubberized concrete at 10%, 12.5%, and 15%. Half of the volume specimens was filled with the rubberized concrete and the other was filled with the normal concrete. Such results can be considered as potential reasons for those experimental results.

4.2. Ultimate Flexural Strength and Strains Capacity

Table 3 presents the ultimate flexural strength, which was obtained from the load-deflection curve of the tested beams and, accordingly, the effectiveness of crumb rubber on Modulus of rupture by a reduction increased the rubber ratio. It was decreased in an average from 16.38 to 14.29 MPa to 13% approximately compared with the controlled specimens. Ahmed et al. [21] reported that the increase in rubber content in concrete mixtures caused a reduction in flexural strength. Table 3 illustrates that the rates of the decreased flexural strength in the hybrid beams structure were better in contrast
with the rubberized concrete without the layer due to the reduction in the rubber contents. Norman [14] reported that the rubberized layer at the top of the hybrid concrete was directly subject to load and absorbed more energy, whereas the bottom of the plain concrete demonstrated more tensile resistance.

### Table 3. Results of the ultimate flexural strength and strain capacity

<table>
<thead>
<tr>
<th>Type of Mix</th>
<th>Flexural strength (Mpa)</th>
<th>Average stress (σcr) (N/mm²)</th>
<th>Average stress (συ) (N/mm²)</th>
<th>Stress at crack (N/mm²)</th>
<th>Average stress at crack (N/mm²)</th>
<th>Strain at crack (%)</th>
<th>Average Strain (%)</th>
<th>Strain capacity of specimen</th>
<th>Longitudinal reinforcement at tension zone</th>
<th>Average Strain (%)</th>
<th>Average Concrete Strain</th>
<th>Average Concrete Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>17.56</td>
<td>16.39</td>
<td>1.55</td>
<td>1.53</td>
<td>3.90</td>
<td>3.64</td>
<td>1.25</td>
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</tr>
<tr>
<td></td>
<td>15.21</td>
<td>1.50</td>
<td>3.38</td>
<td>0.90</td>
<td>4.42</td>
<td>368.00</td>
<td>-1167.00</td>
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<td></td>
<td></td>
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<tr>
<td>RCB1</td>
<td>16.63</td>
<td>15.09</td>
<td>1.39</td>
<td>3.70</td>
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<td>1.22</td>
<td>1.21</td>
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<td>3.01</td>
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<td>597.00</td>
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<td>589.00</td>
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<td>16.32</td>
<td>1.31</td>
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<td>6.92</td>
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<td>698.00</td>
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<td>1614.50</td>
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<td>13.21</td>
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<td>HRCB4</td>
<td>15.96</td>
<td>16.04</td>
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<td>1.14</td>
<td>4.18</td>
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<td>305.00</td>
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<tr>
<td></td>
<td>16.11</td>
<td>1.40</td>
<td>3.58</td>
<td>1.05</td>
<td>4.43</td>
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<td>4.81</td>
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<td>500.00</td>
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<td>5.33</td>
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<td>3.43</td>
<td>1.13</td>
<td>5.26</td>
<td>4.82</td>
<td>1621.00</td>
<td>680.00</td>
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<td>-1522.00</td>
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</table>

Based on Table 3 and Figure 7, similar effects of rubber were observed in stress at ultimate load in the rubberized and hybrid rubberized concrete through dropped results, in general, regarding the contents. On the other hand, the strain capacity of the rubberized beams was improved on average by 39.23% at 15% Cr from the stress-strain curve and 32.6% from concrete strain gauges records. This is considered as an accurate measurement for confirmation. The results of this study were in line with the findings that were reported by Agampodi et al. [26]. Also, the gauges recorded the strain on the longitudinal reinforcement at tension zone during the loading. As displayed in Table 3, the strain of steel bars increased as the rubber content increased. However, the strain was less than $600 \times 10^{-6}$ . This was expected according to what has been reported by previous findings. Therefore, it is confirmed that the strain in the steel bar at the tension zone will be less even though crumb rubber content increases [6]. In the hybrid rubberized concrete beam, the strain by gauges and stress-strain curve at concrete became lower between 1.32% to 2.66% and 19.13% to 30.34%, respectively in contrast with the rubberized concrete at the same rubber proportion (from 10% to 15%). In contrast, the stress became higher between 6.25% and 1.25%. All these can be attributed to the changeable materials’ volume.
Figure 7. Stress-strain curve: a) Rubberized concrete, b) Hybrid rubberized concrete, and Strain gauges: c) Rubberized concrete, d) Hybrid rubberized concrete
4.3. Characteristics of Load-deflection Behaviour

Table 4 shows the characteristics of mid-span deflection of beams, which were monitored by the naked eye and the calculation equations pertaining to the flexural test. As shown in Table 4, the first cracking load decreased as the percentage of Cr increased. For example, the inclusion of 10% Cr in RCB1 has shown a reduction of an average of 8.7% compared with control and decreased to 15.47% at 15%Cr. It was observed that in a hybrid beam, there was not a major improvement that caused the delay when the first cracking load emerged. Similarly, the trend was reported at deflection at first cracking. However, there was a minor effect on first cracking load.

<table>
<thead>
<tr>
<th>Types of Mix</th>
<th>specimens No.</th>
<th>Cracking Load, Per (KN)</th>
<th>Cracking Deflection, X cr (mm)</th>
<th>yield Load, Pyield (KN)</th>
<th>Yield Deflection, Ayield (mm)</th>
<th>Ultimate Load, Pu (KN)</th>
<th>Ultimate Deflection, uult (mm)</th>
<th>Total energy, Etotal (Toughness) (KN.mm)</th>
<th>Stiffness, K(KN/m)</th>
<th>ductility index μ</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>1</td>
<td>46.41</td>
<td>2.50</td>
<td>98.15</td>
<td>7.70</td>
<td>117.05</td>
<td>11.05</td>
<td>366.96</td>
<td>16.01</td>
<td>1.44</td>
</tr>
<tr>
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<td>Average</td>
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<td>2.31</td>
<td>93.23</td>
<td>6.75</td>
<td>109.22</td>
<td>9.95</td>
<td>343.24</td>
<td>16.25</td>
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<td>2.42</td>
<td>82.21</td>
<td>5.31</td>
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<td>10.65</td>
<td>367.26</td>
<td>13.94</td>
<td>2.01</td>
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<td>74.87</td>
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<td>10.68</td>
<td>395.67</td>
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<td>4.84</td>
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<td>8.61</td>
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<td>2.24</td>
<td>82.53</td>
<td>4.19</td>
<td>100.56</td>
<td>9.48</td>
<td>356.77</td>
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<tr>
<td>HRB3</td>
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<td>2.28</td>
<td>90.95</td>
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<td>102.80</td>
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<td>39.41</td>
<td>2.27</td>
<td>84.35</td>
<td>4.11</td>
<td>96.48</td>
<td>9.63</td>
<td>366.51</td>
<td>13.97</td>
<td>2.35</td>
</tr>
</tbody>
</table>

Zemei et al. [27] reported that there was a significant effect of crumb rubber that was added in the reinforced concrete on the ultimate load and deflection rather than a minor effect on first cracking load. Figure 8 shows the ultimate load-deflection curves in an average for the rubberized and hybrid rubberized concrete. The addition of 15% crumb rubber aggregate as a replacement of fine aggregate seemed to increase the ultimate deflection to 39.09% compared with the reinforcement of controlled concrete, whereas the ultimate load demonstrated a reduction occurred at 12.7% in contrast with control. Therefore, it is expected that there will be less strain in steel at tension by increasing the Cr content, which can be attributed to a decrease in the ultimate load [6]. Using 10% crumb rubber in the hybrid concrete beams decreased deflection and increased load at 19.15% and 6.29 %, respectively. These results can be attributed to the use of double-layered and rubber at the top. This was done in order to absorb the load and to minimize flexural strength loss.
4.4. Toughness, Stiffness, and Ductility

The toughness (total energy) listed in Table 4 for each mixture was obtained by measuring the area enclosed by the deflection-curve of the reinforced concrete during the flexural test. Increasing the percentage of Cr has positively affected the ability of the beam to absorb the total energy up to failure and the deformability of the tested beams has increased. In this study, there was an increase in the deformability area for all the tested beams, which showed an increase in toughness. In the rubberized concrete beams, varying from 0% to 15% of crumb rubber percentage significantly increased toughness on average by 24.08%. Mohamed et al. [18] obtained high toughness by adding 5 to 15 % of crumb rubber content with 4.75 mm size into concrete and confirmed that the increase in Cr percentage beyond 15 % reduced toughness prior to failure. Other studies by the same authors confirmed these findings [6]. The hybrid concrete beams at failure absorbed a lower amount of total energy than in the rubberized concrete beams. Such findings of comparison on average, at 10% the rubber content for RCB1 reduced significantly on average from 367.26 to 331.55 for beams HRCB4 as shown in Figure 9. According to Guo et al. [28], this toughness increased when rubber content increased. Other researchers confirmed that the inclusion of crumb rubber in beams helped to increase the chances of the deformability, which improved the energy absorption [6].

In this study, the ductility index (μ) was used to assess the influence of crumb rubber on the deformability of all the tested beams. Generally, the inclusion of the crumb rubber in beams can contribute to improving strain and deformability and enhancing the duality in terms of increase [6, 29]. For instance, increasing the rubber from 0-15 % in the rubberized and hybrid rubberized concrete beams, which increased the ductility by 110.8 % and 35.2%, respectively.
Against this trend, when beams were more ductile and deform by increasing the rubber contents, this resulted in lower stiffness beams, e.g., RCB1 showed slightly lower average values at 13.11 compared with RC at 16.25 KN/mm. On the other hand, hybrid concrete beams were considerably stiffer as clearly illustrated in Table 4.

4.5. Cracking Behaviour

Figure 11 (a, b) and Table 5 show the failure mode of the cracking (numbers, angle, widths, mix, and minim spacing) and of the tested beams. In the rubberized concrete beams, increasing the crumb rubber contents volume from 0% to 15% resulted in an increase in the number of cracking by 37% on average. On the other hand, the cracking numbers decremented in the hybrid beams by 16% on average in contrast with control. This is because of the modules of the elasticity of rubber aggregate that is lower than fine aggregate, which caused an increased number of cracking on the beams until crashing [6, 30]. In contrast, the widths appeared to be narrow by 39.3% at 15% crumb rubber compared with control. However, it became narrow at 36% for the hybrid concrete beams as shown in Figure 10. Mohamed K [6] reported an increase in the numbers of crack and decremented of the width of as Cr content increased.

![Graph showing No. of sperial (No.) & Max of crack width (mm)](image)

**Figure 10. Crack behaviour and its Width Results**

<table>
<thead>
<tr>
<th>Type of Mix</th>
<th>No. of sperial</th>
<th>Average (mm)</th>
<th>Max of crack width (mm)</th>
<th>Average (mm)</th>
<th>Angle of crack at failure degree (deg.)</th>
<th>Average (deg.)</th>
<th>Minimum spacing (mm)</th>
<th>Average (mm)</th>
<th>Maximum spacing (mm)</th>
<th>Average (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>10</td>
<td>9.5</td>
<td>6</td>
<td>5.04</td>
<td>39</td>
<td>39</td>
<td>20</td>
<td>29</td>
<td>141</td>
<td>185</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>5.52</td>
<td></td>
<td>35</td>
<td>37</td>
<td>36</td>
<td>18</td>
<td>17</td>
<td>130</td>
<td>142</td>
</tr>
<tr>
<td>RCB1</td>
<td>11</td>
<td>1.1</td>
<td>4.33</td>
<td>4.44</td>
<td>37</td>
<td>35</td>
<td>15.69</td>
<td>14.57</td>
<td>155</td>
<td>133.5</td>
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<tr>
<td>RCB2</td>
<td>11</td>
<td>11.5</td>
<td>3.74</td>
<td>3.74</td>
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<td>35</td>
<td>13.45</td>
<td>112</td>
<td>155</td>
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</tr>
<tr>
<td></td>
<td>12</td>
<td>3.73</td>
<td></td>
<td>32</td>
<td>31</td>
<td>32</td>
<td>10</td>
<td>9.49</td>
<td>131</td>
<td>126</td>
</tr>
<tr>
<td>RCB3</td>
<td>12</td>
<td>13</td>
<td>3.05</td>
<td>3.36</td>
<td>31</td>
<td>32</td>
<td>9.745</td>
<td>131</td>
<td>121</td>
<td>146</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>3.66</td>
<td></td>
<td>32</td>
<td>32</td>
<td>32</td>
<td>9.49</td>
<td>131</td>
<td>121</td>
<td>146</td>
</tr>
<tr>
<td>HRCB4</td>
<td>9</td>
<td>9.5</td>
<td>4.77</td>
<td>4.75</td>
<td>38</td>
<td>38</td>
<td>16</td>
<td>20.5</td>
<td>137</td>
<td>146</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>4.72</td>
<td></td>
<td>38</td>
<td>38</td>
<td>38</td>
<td>16</td>
<td>20.5</td>
<td>137</td>
<td>146</td>
</tr>
<tr>
<td>HRCB5</td>
<td>9</td>
<td>10.5</td>
<td>4.14</td>
<td>4.03</td>
<td>37</td>
<td>37</td>
<td>17</td>
<td>17.5</td>
<td>132</td>
<td>141.5</td>
</tr>
<tr>
<td></td>
<td>12</td>
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<td>37</td>
<td>37</td>
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<td>17</td>
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<td>141.5</td>
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<tr>
<td>HRCB6</td>
<td>10</td>
<td>11</td>
<td>3.59</td>
<td>3.72</td>
<td>45</td>
<td>45</td>
<td>16</td>
<td>16.7</td>
<td>137</td>
<td>135.5</td>
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<tr>
<td></td>
<td>12</td>
<td>3.85</td>
<td></td>
<td>30</td>
<td>38</td>
<td>17.4</td>
<td>16</td>
<td>16.7</td>
<td>137</td>
<td>135.5</td>
</tr>
</tbody>
</table>

The crack propagation and its spacing for the rubberized and hybrid rubberized concrete of tested beams were measured. According to previous studies [6, 21, 29, 30], when the crumb rubber was added to conventional concrete, it
caused more cracks. This indicates that the spacing between the two cracks will become lower whether for maximum or minimum spacing. For example, in the maximum spacing in RCB3 with the inclusion of 15% of crumb rubber decreased by an average of 23% compared with the conventional concrete (RC). Similarly, trends with minimum spacing were observed; the spacing decremented by 66%. A hybrid, which has crumb rubber at the top of the tested beams will help to enhance crack behaviour and its spacing. Therefore, the maximum and minimum in HRCB4 were slightly enhanced by 3% and 18%, respectively. Further enhancement was observed in HRCB6 with 15% of fine aggregate replacement by crumb rubber aggregate at 8% and 7%, respectively. This has become clearer when it was compared with RCB3.
Figure 11. a) Crack behaviour of controlled and rubberized concrete
5. Conclusion

Fourteen rubberized reinforced concrete beams and hybrid reinforced concrete beams with double layers were investigated under flexural behaviour. The double-layer beam with a rubberized beam on the top was used with a ratio of 10%, 12.5%, and 15% of sand volume replacement and the reinforcement concrete at the bottom. The fresh properties, failure mod, total energy (toughness), stiffness, and ultimate deflection, modulus of rupture, strain capacity and ductility index were studied for all the beams. Based on the experimental results, the following findings were obtained:

- The hybrid beam with double layers has shown better performance in contrast with the rubberized beam in the failure patterns (number of cracks, maximum, and minimum spacing), first cracking load, fresh properties, ultimate deflection load, stiffness, flexural strength, and stress.

- The rubberized beam has improved the ultimate deflection, ductility index, the strain that was obtained by both strains’ gauges (concrete and steel bar), and the stress-strain curve in contrast with the hybrid beam.

- Better performance was achieved based on the results of this comparative study regarding the use of the hybrid beams for a new sustainable application in the construction industry.

6. Funding

The authors would like to acknowledge that this research was funded by Universiti Sains Malaysia’s grant (Cluster). For Polymer Composite: 1001/PKT/8640013.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


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Serviceability Assessment of Continuous Beams Strengthened by SMA Strands under Cyclic Loading

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Received 19 January 2019; Accepted 23 April 2019

Abstract

Since the wide cracks or large deflections can have a significant effect on the appearance of concrete elements and may cause some uncommon behavior, therefore, serviceability of concrete structures requires investigation. The main objective of this paper is to study experimentally the serviceability of continuous reinforced concrete (RC) beams strengthened by Ni-Ti strands. In addition, some building code provisions were used to calculate crack width and deflection. The current study presents the experimental results to verify the accuracy of building codes’ provisions for continuous RC beams strengthened by SMA strands. Although a pattern of smaller width cracks was monitored for strengthened beams, more than 50% of the crack widths were recovered because of super elastic SMA strands. The performance of crack width provisions illustrates an overestimated crack width for SMA RC beams. Moreover, the predicted values for immediate deflections based on building codes provided a good agreement, although the effective reinforcement ratio (steel reinforcement and SMA strands) had a significant effect on immediate deflections of reinforced concrete beams strengthened by SMA strands under service loads.

Keywords: Serviceability; Continuous Beam; Cyclic Loads; Strengthening by Nitinol Strands; Building Codes.

1. Introduction

Nowadays, concrete structures are one of the favorable alternatives in the construction industry and they are considered to satisfy the main criteria of limit states. Well-detailed and properly-erected structures designed by the limit state method will have acceptable probabilities that they will not reach a limit state, will not become unfit for their purpose by collapse and buckling (ultimate limit states), deformation and cracking (serviceability limit states), and therefore, the structure will be durable under environmental conditions over its design life. Some researchers have studied the serviceability requirements, crack width and deflection. Ramos et al. developed and validated a finite element model to study the static and dynamic behavior of a reinforced concrete beam during cracking. A nonlinear behavior was expected at the loading cycle because of cracking. However, upon secondary analysis, when it was loaded again up to the same level, the concrete behaved linearly and so it did not suffer more degradation [1]. Allam et al. investigated building codes formulas and different effective factors for crack width calculations in RC flexural members. Standard codes provisions predicted various values, while Egyptian code underestimated crack width, especially in sections with low reinforcement ratio [2]. Desayi and Ganesan considered a concrete member with a reinforcement bar under tension loading and proposed a new method to calculate crack width. The proposed equation overestimated crack width by 5.1%, while the BS8110 provision underestimated crack width by 18.3% [3]. Rakoczy and Deak theoretically

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http://dx.doi.org/10.28991cej-2019-03091312

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investigated the effects of cracked section on deformation of continuous reinforced concrete beams under service limit state. As a result of the proposed method, the plastic redistribution of moments caused a different moment distribution compared to the one assuming constant stiffness [4]. Yasir Alam et al. experimentally tested three different sizes of RC beams to consider crack width and crack spacing under service loads. The results were more or less in agreement with measured values and calculated ones at low strains and small beam size [5]. Araujo presented a nonlinear model as a reference to verify ACI and CEB methods for calculation of immediate and long-term deflections. Both methods showed good results for the uncracked sections and the cracked ones. However, the ACI method is not reliable for deflection calculations related to creep and shrinkage [6]. Shaaban et al. experimentally investigated the crack pattern and deflection of normal strength concrete (NSC) and high strength concrete (HSC) T-beams under service limit state. It was shown that the flange dimensions played an effective role on the beams’ behavior. Experimental results demonstrated that the crack initiation was delayed and short-term deflection decreased by increasing the flange dimensions [7].

Although crack width and crack spacing have been widely investigated, plastic deformation of steel reinforcement at unloading can be considered as an important disadvantage of concrete structures, especially when they are subjected to earthquakes. Usage of shape memory alloys, may well solve this problem. Shape memory alloys (SMAs) are innovative materials that have the potential to sustain large deformations and to revert to their undeformed shape upon removing the stress (superelasticity) or by heating (shape memory effect). These unique properties and their particular behavior, especially under cyclic loading, marked them as a desirable material. Shape memory alloys (SMAs) can be formed into different shapes, and therefore, they have the capability of serving various functions: (a) in bridge restrainers for reducing the movement of the deck during an earthquake; (b) Base-isolation systems; (c) in concrete structures for reducing of permanent deformation and also leads to re-centering capacity of the structure after devastation, and (d) in steel structures as a part of the connections or bracing [8].

Saiidi et al. cyclically tested eight simply supported reinforced concrete beams under two-point loading. Half of them were reinforced by Ni-Ti rods. Loading and unloading with half yield load increments were performed until achieving the displacement ductility of two. It was found that the SMA reinforcement had the ability of recovering deformation under cyclic loading [9]. Debbarma and Saha experimentally tested eight simply supported beams, which half of them were reinforced by SMA rods to investigate immediate and long-term deflections. The super elasticity of the SMAs increased loading capacity, resulting in decline of the instantaneous and long-term deflections of SMA RC beams were declined [10]. Choi et al. carried out the bending test on small-scale beams reinforced by four different types of SMA fibers to offer a new method for crack closing. Although all types of SMA fibers increased the flexural strength of reinforced beams, paper-wrapped fibers exhibited higher cracking recovery because of sufficient anchoring action [11]. Khaloo et al. numerically studied the effect of different parameters on cyclic behavior of RC beams reinforced by smart rebars. It was shown that using smart rebars reduced the residual displacement of RC beams under cyclic loading [12]. Shahjil et al. carried out three point bending tests on beam specimens in which Ni-Ti fibers were embedded for their self-centering capability. Recoverable deformations were observed under cyclic loading, whereas steel reinforcement rebars could not achieve the small results under similar loading conditions [13]. Nubailah et al. proposed a finite element model to report the behavior of reinforced concrete beams with super elastic shape memory alloys subjected to static loading. SMAs played a positive role on limiting residual displacements and crack propagation. Moreover, SMA RC beams experienced higher yield load and displacement ductility compared to conventional RC beams [14]. Hosseini et al. studied the capability of reinforced concrete structures with shape memory alloys. Copper-based memory alloys and Nickel-based memory alloys were separately used in the stimulated finite element model. It was shown that the rate of general strains and plastic strains in models with Cu-based alloy armatures was higher than those with Ni-based alloy armatures. However, columns with Cu-based alloy armatures experienced less lateral load [15]. Elbahi and Yousef analytically compared the flexural behavior of steel and SMA RC beams during loading and unloading stages, by using a displacement-controlled loading method. The parametric study demonstrated that increasing the SMA bar length reduces the amount of residual displacement and flexural stiffness. Correspondingly, the length of SMA bars played a significant role on the amount of dissipated energy [16].

In spite of various investigations on SMA reinforced concrete sections, their service behavior is currently unknown and needs to be studied. Moreover, since demand of self-compacting concrete (SCC) in the construction industry is growing because of its high workability compared to that of typical vibrated concrete structures, this paper focused particularly on the serviceability of reinforced self-compacting concrete continuous beams strengthened by super elastic SMA strands. As mentioned, previous literature mainly focused on vibrated concrete and not on SCC.

In this study, four continuous beams were experimentally tested under cyclic loading, which half of them were strengthened by SMA strands in sagging and hogging regions. Based on experimental results, the service response of beams are discussed by the following steps: (i) cyclic loading in increments related to yield deflection, which was measured in a monotonic test [17]; (ii) monitoring the behavior of tensile reinforcements, concrete strains and deflections, and (iii) measuring flexural crack width and deflection under service loads. Likewise, experimental results
are compared with different standards. Figure 1 illustrates the research methodology by a flowchart of the steps used in the current study.

![Flowchart of the research methodology](image)

Figure 1. The research methodology

2. Theoretical Serviceability Limit State (SLS) According To Building Codes

2.1. Stress Limitations

Different standards consider service conditions based on the elastic behavior of materials. The limitation of concrete compressive stress is to avoid longitudinal cracks or high level creep where they could result in unacceptable effects on the function of the structure. According to ACI 318M-14 [18], the concrete structures are studied under service loads when the compressive stress in the extreme concrete fiber equals 0.45\(f'_c\). Considering CSA A23.3 code [19], the limit for the concrete compressive stress in the serviceability limit state is set to 0.4\(f'_c\). BS8110 [20] explains that in flexural members, the compressive stress should not exceed 0.4\(f_{cu}\) at the extreme concrete fiber in continuous beams. In EN 1992-2 [21], the compressive stress of concrete is limited to the value 0.6\(f_{ck}\) under rare load combinations and 0.45\(f_{ck}\) under quasi-permanent loads. In addition, EN code considered limitations of reinforcement tensile stress to avoid inelastic strain, unacceptable cracking, or deformation. Therefore, the tensile stress of reinforcements is limited to 0.8\(f_{yk}\) under characteristic load combinations and \(f_{yk}\) under an imposed deformation.

2.2. Deflection and Crack Width Considerations

In general, design requirements for the serviceability limit state (SLS) are presented with emphasis on deflection and cracking under service loads. The following deflection and crack width provisions are drawn from different standards.

2.2.1. Crack Width Provisions

i) ACI code

Based on statistical analysis for flexural crack control in beams, Equation 1 predicts the probable maximum crack width [22].
Further researches and experimental works showed that there is no clear relationship between surface crack widths and corrosion. Therefore ACI code proposed a simplified equation based on maximum bar spacing in lieu of earlier crack rules. ACI 318M-14 presented Equation (2) for crack width control [18].

\[
s = 380 \left( \frac{f_y}{f_y} \right) - 2.5c_c \leq 300 \left( \frac{f_y}{f_y} \right)
\]

(2)

ii) Canadian standard

According to CSA A23.3-14 [19], the spacing between the tension bars was considered as the crack width control. Therefore, flexural bars shall be spaced in the tension zone so that the value z in Equation 3 does not exceed 30000N/mm for interior exposure and 25000N/mm for exterior exposure.

\[
z = f_y \sqrt{d_c / A}
\]

(3)

iii) British standard

Based on general provisions of crack width in BS 8110-1997[20], flexural crack width at a particular point on the beam surface depends on a) the concrete cover b) the distance of the neutral axis from the particular point and c) the average surface strain at the considered point. It is declared that the surface crack width, which is calculated from Equation 4 should not exceed 0.3 mm for the visible members (appearance) and the members in aggressive environments (corrosion).

\[
W_s = \frac{3n_{cr} \varepsilon_{em}}{1 + 2(\frac{d_{cr} \varepsilon_{cr}}{A})}
\]

(4)

It should be noted that the elasticity modulus of concrete in the calculation of strain should be taken as half of the instantaneous value.

iv) Eurocode 2

According to Eurocode2 (EN 1992-1-1) [21], the following Equation 5 is presented for crack width calculations.

\[
W_k = S_{r,\text{max}} (\varepsilon_{sm} - \varepsilon_{cm})
\]

(5)

\[
\text{bar spacing} \leq 5 \left( c + \frac{r}{2} \right) \rightarrow S_{r,\text{max}} = 3.4c + 0.425K_1K_2\phi/\rho_{p,\text{eff}}
\]

(6)

\[
\text{bar spacing} > 5 \left( c + \frac{r}{2} \right) \rightarrow S_{r,\text{max}} = 1.3(h - x)
\]

(7)

2.2.2. Deflection Calculations

i) ACI 318M-14 and CSA A23.3-14

Reinforced concrete members subject to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that could adversely affect the strength or serviceability of the structure. When deflections are computed, deflections that occurred immediately upon loading shall be computed by methods or formulas for elastic deflections.

The immediate midspan deflection (\(\Delta_i\)) of a two-span beam under concentrated loads (Figure 2) can be derived by Equation 8.

\[
\Delta_i = \frac{7PL^3}{768EI_e}
\]

(8)

Figure 2. Schematic of a two-span beam under two concentrated loads

For two-span continuous prismatic members, the effective moment of inertia may be taken as the weighted average of the values obtained from Equation 9 for the critical positive and negative moment section. Hence, the average effective moment of inertia can be calculated by Equation 10.

\[
l_e = l_{cr} + (I_{g} - I_{cr}) (\frac{M_{cr}}{M})^3 \leq I_{g}
\]

(9)

\[
l_{e,\text{ave}} = 0.85l_{em} + 0.15l_{eis}
\]

(10)

The transformed uncracked and cracked section of strengthened beams are shown in Figure 3. Meanwhile, Equations 11-14 were expanded to determine the depth of neutral axis and the moment of inertia for both transformed uncracked and cracked section with SMAs, as follows:
i) Transformed uncracked section

\[ \bar{y}_{tr} = \left( \frac{1}{2}B'h + r'd + d' \right) / (1 + B' + r') \]

\[ I_{tr} = \frac{1}{12}bh^3 + bh \left( \bar{y}_{tr} - \frac{h}{2} \right)^2 + \left( (2n_{st} - 1)A'_{st} \right) (\bar{y}_{tr} - d')^2 + \left( (n_{st} - 1)A_{st} + (n_{SMA} - 1)A_{SMA} \right) (d - \bar{y}_{tr})^2 \]

ii) Transformed cracked section

\[ \bar{y}_{cr} = \left( \sqrt{(1 + r)^2 + 4Bd'(1 + r/\bar{d}' - (1 + r))} - 1 \right) / 2B \]

\[ I_{cr} = \frac{1}{3}b\bar{y}_{cr}^3 + \left( (2n_{st} - 1)A'_{st} \right) (\bar{y}_{cr} - d')^2 + \left( (n_{st}A_{st} + n_{SMA}A_{SMA}) / (2n_{st} - 1) \right) (d - \bar{y}_{cr})^2 \]

Where:

\[ B = \frac{b}{2(2n_{st} - 1)A'_{st}} \]

\[ r = \frac{(n_{st}A_{st} + n_{SMA}A_{SMA})}{(2n_{st} - 1)} \]

\[ B' = \frac{bh}{(2n_{st} - 1)A'_{st}} \]

\[ r' = \frac{[(n_{st} - 1)A_{st} + (n_{SMA} - 1)A_{SMA}] / (2n_{st} - 1)}{A'_{st}} \]

![Figure 3. Cross section of strengthened beams: (a) typical section (b) transformed uncracked section (c) transformed cracked section](image)

ii) British standard

According to BS 8110-997 [20], the deflected shape of a member is related to the curvatures, and thus, deflections may be determined by calculating the curvatures at successive sections along the member and using a numerical integration technique. Alternatively, the simplified Equation 15 can be used for calculating deflections based on the curvature.

\[ \Delta = K L^2 \frac{1}{r} \]

It should be noted that K can be determined by using Equation 16 for the bending moment diagram of continuous beams under concentrated loads (Figure 4).

\[ K = 0.083 (1 - \frac{M_{a} + M_{b}}{4M_{c}}) \]

![Figure 4. Continuous beam under concentrated loads: (a) loading condition, (b) bending moment diagram](image)

iii) Eurocode 2

In EN 1992-1-1 [21], the method of assessing deflections is to compute the curvatures at frequent sections along the member and then calculate the deflection by numerical integration. According to EN, in most cases it will be acceptable to compute the deflection twice, assuming the whole member to be in the uncracked and fully cracked condition in turn, and then interpolate using Equation 17.
\[
\alpha = \xi \alpha_{11} + (1 - \xi) \alpha_1 \\
\xi = 1 - \beta \left( \frac{M_{cr}}{M} \right)^2
\]  

(17)  

(18)

3. Experimental Program

3.1. Details of Beam Specimens

Four two-span continuous beams were experimentally tested under cyclic loading up to failure. The beam specimens were designed and casted in two sets of strengthened beams by SMA strands in critical tension regions (BN1-Nm and BN2-Nm) according to the flexural moment diagram and non-strengthened beams (BN1-S and BN2-S), and control beams with just conventional steel reinforcements. The numbers 1 and 2 represent the group of beams according to the percentage of tensile bars. In other words, the beams were entitled BN1 are reinforced by four steel bars of \( \Phi 8 \) at top and bottom, while four steel bars of \( \Phi 10 \) are used at top and bottom of the other two beams were named BN2. Also, one additional tensile bar of \( \Phi 8 \) is added to the beam section at central support. Beam dimensions and reinforcement details are shown in Figure 5 and summarized in Table 1.

Table 1. Beam reinforcement details

<table>
<thead>
<tr>
<th>Group number</th>
<th>Beam type</th>
<th>Steel bar</th>
<th>Additional steel bar</th>
<th>SMA strand</th>
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<th>Section b-b</th>
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<td>1</td>
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<td>4( \Phi 8 )</td>
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<td>None</td>
<td>0.0054</td>
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<tr>
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<td>BN1-Nm</td>
<td>4( \Phi 8 )</td>
<td>None</td>
<td>3( \text{strands}_{x} ) ((l=200))</td>
<td>0.0054</td>
<td>0.0054</td>
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<tr>
<td>2</td>
<td>BN2-S</td>
<td>4( \Phi 10 )</td>
<td>1( \Phi 8 ) ((l=500))</td>
<td>None</td>
<td>0.0112</td>
<td>0.0086</td>
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<tr>
<td></td>
<td>BN2-Nm</td>
<td>4( \Phi 10 )</td>
<td>1( \Phi 8 ) ((l=500))</td>
<td>2( \text{strands}_{x} ) ((l=250))</td>
<td>0.0112</td>
<td>0.0086</td>
</tr>
</tbody>
</table>

3.2. Material Properties

The ASTM standard presents some codes considering the required tests for SMA alloys, especially nickel-titanium superelastic materials. The F2004-05 code suggests the differential scanning calorimatory (DSC) method to determine transformation temperatures of superelastic Nitinol materials [23]. The DSC method was implemented and the temperature of phase transformations were measured. According to the DSC diagram given in Figure 6a, the austenite phase will begin at the temperature \( A_S = 0^\circ C \) and phase transformation will be completed after obtaining the temperature \( A_f = 28^\circ C \) and so the SMA will be completely austenitic. In addition, while it is in a high temperature austenite phase and the material cools down, the austenite to martensite phase transformation will begin at the temperature \( M_S = 26^\circ C \) and will become entirely martensitic whenever the temperature reaches \( M_f = -7.5^\circ C \). The most reliable method for
stress-strain relationship of Nitinol wires are provided in F2516-07 [24]. Figure 6b shows the derived stress-strain diagram of SMA wires by pulling the sample to 6% strain, unloading to less than 7MPa stress, and then pulling up to failure.

![Stress-strain diagram](image)

**Figure 6.** Nitinol wire properties: (a) DSC thermogram; (b) stress-strain diagram

It should be noted that a special machine was used for twisting seven wires to a strand (Figure 7). Likewise, a tension test was carried out on steel reinforcements to determine the required properties. The properties of steel reinforcement and SMA wires such as yield and ultimate strengths and Young’s modulus are provided in Table 2. The tested beams were casted with normal strength self-compacting concrete (SCC), while the average of four cylinder compressive strength of concrete at 28 days after casting ($f'_c$) are reported in Table 3.

![Twisting machine](image)

**Figure 7.** Twisting machine
Table 2. Reinforcement material properties

<table>
<thead>
<tr>
<th>Type</th>
<th>( f_y (\text{MPa}) )</th>
<th>( f_u (\text{MPa}) )</th>
<th>( E (\text{GPa}) )</th>
</tr>
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<td>371.7</td>
<td>534.0</td>
<td>199</td>
</tr>
<tr>
<td>( \Phi_{10} )</td>
<td>323.9</td>
<td>487.7</td>
<td>197</td>
</tr>
<tr>
<td>( \Phi_{12} )</td>
<td>324.3</td>
<td>479.4</td>
<td>209</td>
</tr>
<tr>
<td>Nitinol wire of 0.46 mm dia.</td>
<td>502.45</td>
<td>1635.56</td>
<td>37.7</td>
</tr>
</tbody>
</table>

Table 3. SCC material properties

<table>
<thead>
<tr>
<th>Beams specimen</th>
<th>( f_y (\text{MPa}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>BN1-S</td>
<td>43.51</td>
</tr>
<tr>
<td>BN1-Nm</td>
<td>36.96</td>
</tr>
<tr>
<td>BN2-S</td>
<td>43.56</td>
</tr>
<tr>
<td>BN2-Nm</td>
<td>40.65</td>
</tr>
</tbody>
</table>

3.3. Test Setup and Procedure

The test loading procedure of the beams is shown in Figure 8. All specimens were set up as two-span continuous which were loaded by a hydraulic jack. Some electrical strain gauges and DEMEC gauges were attached to different locations of the steel bars and along the height of the beams to monitor the behavior of the specimens during the test (see Figure 5). Meanwhile, vertical deflections of beams at mid-span and central support were recorded by linear variable displacement transducers (LVDTs). A 0.02 mm accurate microscope was implemented to observe the crack widths at every loading and unloading step. The test was conducted cyclically using "displacement control" method. According to ATC-24 [17], the yield values of displacement (\( \Delta y \)) were measured from a monotonic loading test. A stepwise displacement cycle is recommended to be applied; which was started as 0.33\( \Delta y \) to 1\( \Delta y \) in increments of 0.33\( \Delta y \) and was continued in increments of 1\( \Delta y \) up to the end [17].

![Figure 8. Test setup](image_url)

4. Test Results and Observations

4.1. Cracking Moment

The beams were continuously monitored during the test. While the first visible crack appeared at mid-span, the corresponding force was recorded as the cracking load (\( P_{cr} \)) and the experimental cracking moment (\( M_{exp} \)) for all tested beams were determined. The cracking moments were also calculated theoretically using Equation 19. It is obvious that the cracking moment of a reinforced concrete member is related to the flexural tensile strength, which is proportional to the compressive strength of concrete. According to Eurocode 2, the flexural tensile strength depends on the mean axial tensile strength (\( f_{ctm} \)); in other words, it is a function of the compressive strength.

\[
M_{cr} = \frac{f_y A_t}{\gamma_r} \quad (19)
\]

\[
f_y = 0.62 \sqrt{\frac{f_c}{\gamma_r}} \quad \text{ACI 318M-14 [18]} \quad (19a)
\]

\[
f_y = 0.3 \sqrt{\frac{f_c}{\gamma_r}} \quad \text{CSA A23.3-14 [19]} \quad (19b)
\]

\[
f_y = \max \left( 1.6 - \frac{h}{1000}, f_{ctm} \right) \quad \text{EN 1992-1-1 [21]} \quad (19c)
\]
Table 4 provides experimental and theoretical cracking moments for all the tested beams. It is considered that beams strengthened by SMA strands experienced higher cracking moment compared to that of the control beams. In addition, the enhancement ratio of cracking moment ($\gamma$) shows an increase of 38% and 15% in cracking moment of strengthened beams, BN1-Nm and BN2-Nm, respectively. Analyzing the ratio of theoretical to experimental cracking moment ($M_{cr}/M_{exp}$) in Table 4, it is understood that EN 1992-1-1 code had the most conservative prediction of cracking moment with an average theoretical to experimental ratio of 2.77 for the strengthened beams. By contrast, the mean value of theoretical to experimental ratio of cracking moments for beams strengthened by SMA is about 1.056 for CSA, which shows that CSA standard predicts the cracking moment of strengthened beams, unconservatively.

<table>
<thead>
<tr>
<th>Beam</th>
<th>$M_{exp}$ (KN-m)</th>
<th>$\gamma$</th>
<th>ACI 318M-14</th>
<th>CSA A23.3-14</th>
<th>EN 1992-1-1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_{cr}$ (KN-m)</td>
<td>$M_{cr}/M_{exp}$</td>
<td>$M_{cr}$ (KN-m)</td>
<td>$M_{cr}/M_{exp}$</td>
<td>$M_{cr}$ (KN-m)</td>
</tr>
<tr>
<td>BN1-S</td>
<td>0.981</td>
<td>1.00</td>
<td>2.474</td>
<td>2.522</td>
<td>1.201</td>
</tr>
<tr>
<td>BN1-Nm</td>
<td>1.138</td>
<td>1.38</td>
<td>2.771</td>
<td>2.435</td>
<td>1.345</td>
</tr>
<tr>
<td>BN2-S</td>
<td>1.021</td>
<td>1.00</td>
<td>2.377</td>
<td>2.328</td>
<td>1.157</td>
</tr>
<tr>
<td>BN2-Nm</td>
<td>1.178</td>
<td>1.15</td>
<td>2.252</td>
<td>1.912</td>
<td>1.096</td>
</tr>
</tbody>
</table>

4.2. Experimental Stresses under Service States

Standard provisions for allowable stresses (see Section 2.1) were implemented to verify the serviceability state. The permissible strain was calculated according to the elastic behavior of materials under service loads. The material strains were continuously recorded during the test, and therefore, the service load was determined. The results for concrete and steel reinforcement stresses under service loads are summarized in Table 5. It was found that steel stress limitations are critical for the control beam BN1-S, and so, the corresponding loads are measured as the service load in which steel reinforcements obtain their allowable elastic levels. By contrast, the other beams mostly reached their serviceability state limit under concrete stress limitations, and thus, the load was considered as the service load. Meanwhile, all the mentioned codes predict roughly the same service load for the tested beams. In the strengthened beams (BN1-Nm and BN2-Nm), SMA strands were only used in critical tension regions, and therefore, as expected, they had no specific effect on compressive concrete stress. However, the tensile stress in steel reinforcements under service loads declined significantly compared to that of the control beams.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Building code</th>
<th>Load (KN)</th>
<th>Loading cycle</th>
<th>$f_c$ (MPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_y/f_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BN1-S</td>
<td>ACI 318M-14</td>
<td>65.13</td>
<td>C3</td>
<td>371.7</td>
<td>1</td>
<td>13.80</td>
</tr>
<tr>
<td></td>
<td>CSA A23.3-14</td>
<td>65.13</td>
<td>C3</td>
<td>371.7</td>
<td>1</td>
<td>13.21</td>
</tr>
<tr>
<td></td>
<td>BS 8110</td>
<td>65.13</td>
<td>C3</td>
<td>371.7</td>
<td>1</td>
<td>17.06</td>
</tr>
<tr>
<td></td>
<td>EN 1992-1-1</td>
<td>62.11</td>
<td>C2</td>
<td>297.35</td>
<td>0.8</td>
<td>10.03</td>
</tr>
<tr>
<td>BN1-Nm</td>
<td>ACI 318M-14</td>
<td>59.68</td>
<td>C4</td>
<td>245.96</td>
<td>0.66</td>
<td>16.63</td>
</tr>
<tr>
<td></td>
<td>CSA A23.3-14</td>
<td>54.87</td>
<td>C3</td>
<td>257.11</td>
<td>0.69</td>
<td>14.77</td>
</tr>
<tr>
<td></td>
<td>BS 8110</td>
<td>50.92</td>
<td>C4</td>
<td>201.00</td>
<td>0.54</td>
<td>16.79</td>
</tr>
<tr>
<td></td>
<td>EN 1992-1-1</td>
<td>57.81</td>
<td>C3</td>
<td>259.30</td>
<td>0.70</td>
<td>16.65</td>
</tr>
<tr>
<td>BN2-S</td>
<td>ACI 318M-14</td>
<td>122.21</td>
<td>C4</td>
<td>302.00</td>
<td>0.93</td>
<td>19.60</td>
</tr>
<tr>
<td></td>
<td>CSA A23.3-14</td>
<td>115.79</td>
<td>C4</td>
<td>288.81</td>
<td>0.89</td>
<td>17.43</td>
</tr>
<tr>
<td></td>
<td>BS 8110</td>
<td>103.20</td>
<td>C4</td>
<td>323.90</td>
<td>1</td>
<td>19.13</td>
</tr>
<tr>
<td></td>
<td>EN 1992-1-1</td>
<td>107.72</td>
<td>C4</td>
<td>259.12</td>
<td>0.8</td>
<td>19.21</td>
</tr>
<tr>
<td>BN2-Nm</td>
<td>ACI 318M-14</td>
<td>83.29</td>
<td>C5</td>
<td>212.17</td>
<td>0.66</td>
<td>18.28</td>
</tr>
<tr>
<td></td>
<td>CSA A23.3-14</td>
<td>76.45</td>
<td>C5</td>
<td>188.33</td>
<td>0.58</td>
<td>16.27</td>
</tr>
<tr>
<td></td>
<td>BS 8110</td>
<td>68.60</td>
<td>C4</td>
<td>243.69</td>
<td>0.75</td>
<td>18.32</td>
</tr>
<tr>
<td></td>
<td>EN 1992-1-1</td>
<td>68.97</td>
<td>C4</td>
<td>259.06</td>
<td>0.8</td>
<td>17.95</td>
</tr>
</tbody>
</table>

Note: $f'_c$ is assumed as specified compressive strength of concrete ($f'_c$) in ACI and CSA codes, the cube strength of concrete ($f_c$) in BS8110 standard and characteristic cylinder strength ($f_{ck}$) in Eurocode 2.

4.3. Crack Results

4.3.1. Cracking Propagation

Following the guidelines for cyclic testing [17], first, all specimens were loaded up monotonically. The first visible cracks appeared at midspan during the monotonic test. At the end of this step, the cracks were completely recovered in...
BN1-Nm and BN2-Nm. By contrast, control beams BN1-S and BN2-S were capable of recovering just 50% of the first crack widths. Loading cyclically, existing midspan cracks became wider and some new ones appeared at both the point load and central support. Figure 9 shows the crack propagations of the tested beams at the service state. As shown in the figure, the number of midspan cracks was more than that of the central support. Whereas SMA RC beams tend to develop cracks of smaller width compared to the control beams.

![Figure 9. Service crack propagation: (a) BN1-S; (b) BN2-S (c) BN1-Nm (d) BN2-Nm](image)

Table 6 provides the initial and maximum flexural cracking characteristics for the beams tested under cyclic loading. In the conventional RC beams, BN1-S and BN2-S, the first visible flexural crack appeared at approximately 19.43 KN and 12.8 KN, respectively. At these load levels, both BN1-S and BN2-S had a crack width of about 0.08 mm. The superelastic property of SMA strands resulted in smaller width cracks in less cracking load for strengthened beams, BN1-Nm and BN2-Nm. The relative ratio of loads (α) shows a decrease of about 21% and 8% in the cracking load of SMA RC beams, BN1-Nm and BN2-Nm, compared to the corresponding control beams. Considering the reinforcing details of the tested beams, it was found that the increase in reinforcement ratio of SMA RC beams caused less decrease in the amount of cracking load compared to that of conventional RC beams. Moreover, the strengthened beams were found to capable of recovering the initial crack width. At the unloading step, the initial crack in beam BN1-Nm was completely recovered; and the residual crack width in BN2-Nm was negligible (less than 0.01 mm). Service crack characteristics such as maximum crack width (\(w_{cr,max}\)), residual crack width (\(R_{cr}\)) and the recovery capacity of crack width are also reported in Table 6. It can be seen that the maximum flexural crack width in the strengthened beams is less than that of nonstrengthened ones. Meanwhile, SMA RC beams recovered approximately 70% and 87% of the crack width under service load. However, less than 50% of crack width were recorded in the control beam.

<table>
<thead>
<tr>
<th>Beam type</th>
<th>Initial flexural crack</th>
<th>Maximum service crack</th>
<th>Recovery capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(P_{cr}(KN))</td>
<td>(W_{cr}(mm))</td>
<td>(R_{cr}(mm))</td>
</tr>
<tr>
<td>BN1-S</td>
<td>19.43</td>
<td>0.08</td>
<td>0.02</td>
</tr>
<tr>
<td>BN1-Nm</td>
<td>15.41</td>
<td>0.04</td>
<td>0</td>
</tr>
<tr>
<td>BN2-S</td>
<td>12.8</td>
<td>0.08</td>
<td>0.04</td>
</tr>
<tr>
<td>BN2-Nm</td>
<td>11.73</td>
<td>0.04</td>
<td>&lt; 0.01</td>
</tr>
</tbody>
</table>

4.3.2. Assessment of Crack Width Provisions

The crack width provisions of the mentioned standards are applied to the tested specimens and the results are compared with experimental data in Table 7 and Figure 10. Generally, the values of crack widths showed a large scatter among the code equations. As shown, the values predicted by ACI 318M-14 are the highest among those of other codes, although BS 8110 and EN 1992-1-1 mostly predicted similar results for service crack widths. The results obtained from the equations propose an underestimated service crack width for the beam BN2-S, the section reinforced with the ratio of 0.86%. However, ACI 318M-14 was found to provide the best correlation with the experimental service crack width, with the predicted to experimental value of 0.87. In general, ACI 318M-14 predicts more realistic values of service crack width compared to those by other codes.

The values of predicted to experimental ratio are well ranged from -22% to 15% for control beams while those of SMA RC beams indicate that standards provisions overestimated the value of service crack widths for RC beams strengthened by SMA strands. It is a predictable finding because the standards crack width equations were just formulated for conventional reinforced concrete beams. Although a much more logical finding can be achieved by a wide range of experimental data, it is evident that crack width provisions of building codes must be revised for a substantial decline in service crack width of SMA RC beams.
Service crack width of SMA RC beams

\[ \rho_{\text{eff}} = 0.56\% \]

\[ W_{\text{SMA-RC}} = 0.28W_{\text{RC}} \]

\[ R^2 = 0.99 \]

Service crack width of SMA RC beams

\[ \rho_{\text{eff}} = 0.88\% \]

\[ W_{\text{SMA-RC}} = 0.20W_{\text{RC}} \]

\[ R^2 = 0.98 \]

Figure 10. Comparison of maximum crack width based on building codes with experimental maximum crack width under service loads

Table 7. Values of service crack widths based on standards and experimental results

<table>
<thead>
<tr>
<th>Tested specimen</th>
<th>ACI 318M-14</th>
<th>BS 8110</th>
<th>EN 1992-1-1</th>
<th>Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Eq. (1)</td>
<td>pred./exp.</td>
<td>Eq. (4)</td>
<td>pred./exp.</td>
</tr>
<tr>
<td>BN1-S</td>
<td>0.230</td>
<td>1.15</td>
<td>0.202</td>
<td>1.01</td>
</tr>
<tr>
<td>BN1-Nm</td>
<td>0.164</td>
<td>2.73</td>
<td>0.147</td>
<td>2.45</td>
</tr>
<tr>
<td>BN2-S</td>
<td>0.192</td>
<td>0.87</td>
<td>0.171</td>
<td>0.78</td>
</tr>
<tr>
<td>BN2-Nm</td>
<td>0.176</td>
<td>4.40</td>
<td>0.158</td>
<td>3.95</td>
</tr>
</tbody>
</table>

In SMA RC beams, conventional steel bars and SMA strands were used simultaneously which caused new conditions compared to RC beams with just steel bars. Some particular factors such as steel reinforcement stress and effective reinforcement ratio (conventional steel bars plus SMA strands) affected the crack width in SMA RC beams, which caused narrow cracks to appear. The relationship between experimental service crack width in conventional RC beams and SMA RC beams are shown in Figure 11. The most significant feature of the graph is a dramatic decrease in crack widths under service loads for SMA RC beams. Two groups of beams with the effective reinforcement ratios of 0.56% and 0.88% were experimentally tested. A linear relationship was obtained with the value of service crack width in SMA RC beams and conventional RC beams. The value of service crack width in RC beams strengthened by SMA strands was about 28% and 20% of the corresponding value for RC beams with the effective reinforcement ratio of 0.56% and 0.88%, respectively. On the other hand, the values of the crack widths in SMA RC beams are roughly less than 30% of crack widths in conventional RC beams. Meanwhile, more effective ratio of tension reinforcements caused more narrow cracks in the SMA RC beams.

Figure 11. The relationship between observed service crack width in conventional RC beams and SMA RC beams
4.4. Deflection Considerations

4.4.1. Experimental Deflection Behavior

For every loading cycle, the mean value of beam deflection obtained from LVDTs at two midspans are calculated for deflection assessment. The relationship between total applied load and midspan deflection for all tested beams are plotted in Figure 12. Each curve represents the pushover of average midspans’ deflection under service loads. It can be seen that maximum service deflection and its residual value for control beams are significantly more than those of the tested beams strengthened by SMA strands. Maximum deflection values of about 1.15 mm and 1.88 mm were recorded for control beams BN1-S and BN2-S corresponding to 83 KN and 130 KN, respectively. Whereas the strengthened beams, BN1-Nm and BN2-Nm, deformed up to approximately 0.62 mm and 1.56 mm corresponding to 58 KN and 100 KN, respectively. While unloading, SMA RC beams BN1-Nm and BN2-Nm were capable of recovering roughly 86% and 69% of maximum service deflection, respectively. However, approximately 58% and 46% of the maximum deflection under service loads recovered in the control beams BN1-S and BN2-S.

As expected, the crack pattern along the beam is different. In turn, the flexural stiffness (EI) has different values based on whether the considered section is cracked or uncracked. The variation of flexural stiffness is directly related to that of the moment of inertia (I); therefore, the ratio of \( I_e/I_g \) is used to study the flexural stiffness variation of the tested beams. Figure 13 illustrates the relationship between \( I_e/I_g \) and \( M/M_{cr} \) at midspan and central support of all the beams. As shown, the \( I_e/I_g \) trend of the SMA RC beams is roughly similar to that of the corresponding control beams. However, the strengthened beams experienced lower flexural stiffness for a specific \( M/M_{cr} \). There was a significant decline in the ratio \( I_e/I_g \) of cracked specimens until values of \( M/M_{cr} \) are less than 2.5 and 1.5 at midspan and central support, respectively. From this point onwards, although the applied moment increased, the ratio \( I_e/I_g \) leveled out.

![Figure 12. Applied load versus experimental deflection](image1)

![Figure 13. Variation of \( I_e/I_g \) versus \( M/M_{cr} \) at midspan and central support](image2)
4.4.2. Evaluation of Deflection Provisions

According to the mentioned standard provisions (see Section 2.2.2), immediate deflection is calculated at the service limit state. Table 8 provides the predicted deflection ($\Delta_{pred}$), experimental deflection ($\Delta_{exp}$) and the relative ratio of midspan deflection in strengthened beam to that of the corresponding RC beam ($\gamma$). Experimental midspan deflections were measured as 0.45mm and 0.97mm in BN1-Nm and BN2-Nm, respectively, and were significantly less than those of the corresponding RC beams. The values of $\gamma$ demonstrate a decrease of about 50% in midspan deflection of strengthened beams by the SMA strands. In fact, the substantial decline of midspan deflection for SMA RC beams compared with conventional ones are mainly due to their higher displacement ductility, because of their strengthening with superelastic Ni-Ti strands. The values of deflection ratios $\Delta_{pred}/\Delta_{exp}$ ($\beta$) are reported in Table 8, and are plotted in Figure 14 for all tested beams in terms of different building codes. As can be seen, the predicted values of ACI 318M-14 and CSA A23.3-14 are approximately the same. Likewise, these two codes predicted the highest instantaneous deflection for beams compared to other building codes. Because of the superelastic property of SMAs, the midspan displacement in SMA RC beams declined compared to that of control beams. Although the decrease of immediate deflection in strengthened beams is clearly obtained from code provisions, the building codes predicted deflection of SMA RC beams, differently. Code provisions conservatively predicted the immediate deflection of beam BN1-Nm ($\rho_{eff} = 0.56\%$) with the value of $\beta$ ranged between 1.62 and 2.53, whereas the range of $\beta$ from 0.90 to 1.03 showed an unconservative prediction of instantaneous deflection for beam BN2-Nm ($\rho_{eff} = 0.88\%$). The lower ratio of effective reinforcement demonstrated the more conservative predicted deflection for SMA RC beams. Hence, code provisions for RC beams strengthened by SMAs must be revised with the effective reinforcement ratio in mind. Further tests on this subject are essential.

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Delta_{exp}(mm)$</th>
<th>$\gamma$</th>
<th>ACI 318M-14 $\Delta_{pred}(mm)$</th>
<th>$\beta$</th>
<th>CSA A23.3-14 $\Delta_{pred}(mm)$</th>
<th>$\beta$</th>
<th>BS 8110 $\Delta_{pred}(mm)$</th>
<th>$\beta$</th>
<th>EN 1992-1-1 $\Delta_{pred}(mm)$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BN1-S</td>
<td>1.09</td>
<td>1.00</td>
<td>1.33</td>
<td>1.22</td>
<td>1.39</td>
<td>1.28</td>
<td>1.10</td>
<td>1.01</td>
<td>1.23</td>
<td>1.13</td>
</tr>
<tr>
<td>BN1-Nm</td>
<td>0.45</td>
<td>0.41</td>
<td>1.09</td>
<td>2.42</td>
<td>1.14</td>
<td>2.53</td>
<td>0.92</td>
<td>2.04</td>
<td>0.73</td>
<td>1.62</td>
</tr>
<tr>
<td>BN2-S</td>
<td>1.79</td>
<td>1.00</td>
<td>1.95</td>
<td>1.09</td>
<td>2.03</td>
<td>1.13</td>
<td>1.57</td>
<td>0.88</td>
<td>1.39</td>
<td>0.78</td>
</tr>
<tr>
<td>BN2-Nm</td>
<td>0.97</td>
<td>0.54</td>
<td>0.96</td>
<td>0.99</td>
<td>1.00</td>
<td>1.03</td>
<td>0.87</td>
<td>0.90</td>
<td>0.89</td>
<td>0.92</td>
</tr>
</tbody>
</table>

Figure 14. Ratio of predicted to experimental immediate deflection

5. Conclusions

The serviceability of two-span reinforced concrete beams strengthened by SMA strands under cyclic loading was investigated experimentally. Different building codes were also used to assess the service response of strengthened beams such as crack width and deflection. The main results were obtained as follows:

- Unique superelasticity of Ni-Ti strands influenced the concrete pre-cracking stage. Therefore, the cracking moment of RC beams strengthened by SMA strands increased significantly up to 40%. However, theoretical equations predicted higher values of cracking moment compared to experimental data.
- Material strains were monitored continuously during the test to determine service loads. In addition, different building codes were implemented to specify the allowable stress of materials (concrete and steel reinforcement) at service limit state. It was found that RC beams mostly reached to the serviceability limit state under steel reinforcement limitations. By contrast, using SMA strands in strengthened beams caused a substantial decline in steel reinforcement stresses and so concrete stress was highlighted as the service level.
• Considering crack propagations under service loads, smaller width cracks were developed in SMA RC beams compared with control beams. Likewise, SMA RC beams were capable of recovering more than 50% of the service crack widths. On the other hand, theoretical crack widths illustrate that crack width provisions of building codes overestimated the crack widths under service loads for SMA RC beams.

• Experimental deflections of the tested beams showed that the maximum midspan deflection of SMA RC beams was substantially less than that of the control beams. Moreover, RC beams strengthened by SMA strands were able to recover up to 90% of the maximum service deflections.

• Although the $I_e/I_g$ trend of SMA RC beams is roughly similar to that of corresponding control beams, the ratio $I_e/I_g$ of cracked beams decreased substantially for the values $M/M_{cr}$ up to 2.5 and 1.5 at midspan and central support, meaning that the tested beams experienced more cracks. However, for higher values of applied moment, the ratio $I_e/I_g$ remained approximately at the same level.

• Comparison between theoretical deflections based on building codes and experimental data demonstrated a good agreement for the tested beams. However, the effective reinforcement ratio (steel reinforcement and SMA strands) had a significant effect on immediate deflections of reinforced concrete beams strengthened by SMA strands under service loads.

6. Notations

The following symbols are used in this paper:

- $A$ = Effective tension area of concrete surrounding the flexural tension reinforcement ($= \frac{2 d c b n}{n}$)
- $A_{st}$ = Area of longitudinal tension reinforcement, $mm^2$
- $A_{ct}$ = Area of longitudinal compression reinforcement, $mm^2$
- $A_{SMA}$ = Area of longitudinal SMA strands, $mm^2$
- $a_{cr}$ = Distance from the particular point to the surface of the nearest longitudinal bar, $mm$
- $b$ = Width of beam, $mm$
- $C$ = Cover to the longitudinal reinforcement, $mm$
- $C_c$ = The least distance from reinforcement surface to the tension concrete face, $mm$
- $C_{min}$ = Minimum cover to tension bars, $mm$
- $d$ = Effective depth to the centroid of the outer layer of reinforcement, $mm$
- $d_c$ = Thickness of cover from the extreme tension fiber to the closest bar, $mm$
- $E_c$ = Elasticity modulus of concrete, MPa
- $E_s$ = Elasticity modulus of steel reinforcement, MPa
- $E_{SMA}$ = Elasticity modulus of SMA strands, MPa
- $f_c$ = Design service stress in concrete, MPa
- $f_{ctm}$ = Mean value of axial tensile strength of concrete, MPa
- $f_{cu}$ = Characteristic compressive cubic strength of concrete at 28 days, MPa
- $f_r$ = Modulus of rupture of concrete, MPa
- $f_t$ = Tensile stress in reinforcement under service loads, MPa
- $f_y(f_{ya})$ = Characteristic yield strength of reinforcement, MPa
- $f_u$ = Ultimate strength of reinforcement, MPa
- $f_y(f_{ya})$ = Characteristic compressive cylinder strength of concrete at 28 days, MPa
- $f_{ck}$ = Characteristic compressive cylinder strength of concrete at 28 days, MPa
- $h$ = Overall depth of the beam, $mm$
- $I_{cr}$ = Moment of inertia of cracked section transformed to concrete, $mm^4$
- $I_e$ = Effective moment of inertia, $mm^4$
- $I_{avg}$ = Midspan and inner support average moment of inertia, $mm^4$
- $I_{sis}$ = Inner support effective moment of inertia, $mm^4$
- $I_m$ = Midspan effective moment of inertia, $mm^4$
- $I_g$ = Moment of inertia of gross concrete section about centroid axis, $mm^4$
- $I_{cr}$ = Moment of inertia of uncracked section transformed to concrete, $mm^4$
- $K$ = Constant depends on the shape of the bending moment diagram
- $K_1$ = Coefficient which takes account of the bond properties of the bonded reinforcement
- $K_2$ = Coefficient which takes account of the distribution of strain

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7. Acknowledgments

The authors acknowledge MIB dental company for providing superelastic Ni-Ti wires, and also express their gratitude to the faculty of Civil Engineering at Shahid Bahonar University of Kerman for providing the laboratory test facilities.

8. Conflict of Interest

The authors declare no conflict of interest.

9. References


Experimental Investigation for Effects of Mini-piles on the Structural Response of Raft Foundations

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Received 13 March 2019; Accepted 9 May 2019

Abstract

Mini-piles made their debut as a cost-effective way to stabilize the historical structures. Recently, mini-piles have increased in popularity all over the world and are being used for bridges, buildings, slope stability, antenna towers, and residential construction. This paper presents the preparing, executing, data acquisition, and result presentation for an experimental work concerns with five scale-down mini-piled raft foundation models. All models were prepared to study the effectiveness of the mini-piled raft foundation in reducing the settlement and the bending moments. Five tests have been achieved. The reference first test includes a raft foundation with 15mm thickness. Second, third, and fourth tests are mini-piled raft foundations with five mini-piles and with thicknesses of 15 mm, 10 mm, and 8mm respectively. Finally, the fifth test dealt with a single mini-pile 178mm in length and 6mm in diameter. It has been adopted to investigate the reference behavior of the single mini-pile. When they were used, the piles have 42 mm center to center distances. A scale-down factor of 1/45, a sandy soil with with \( \phi \) of 40\(^{\circ} \), and relative density of 60% have been considered in all tests. Test results indicated a 45% decrease in settlement for 15mm mini-piled raft foundation comparing with the reference 15mm raft foundation. Moreover, there is no significant difference in settlement between 15mm mini-piled raft foundation comparing with the 10mm and 8mm thick mini-piled raft foundations. Regarding to the bending moments, they decrease at the mid and edge of the 15mm mini-piled raft foundation comparing to those of the reference raft foundation. It has also been noted that the moments are inversely proportional to the thickness of the piled raft foundations. With respect to the mini-piles, it has been found that most of the pile axial loads are transferred to the underneath soil through friction and this friction increases as the raft thickness decreases.

Keywords: Mini-Pile; Raft Foundation; Sandy Soil; Settlement; Bending Moment; Friction.

1. Introduction

A mini-pile is a drilled pile with a diameter equivalent to or less than 300 \( mm \) and lengths up to 30 \( m \). It is having a small diameter to length ratio so that it is slender in nature, most of the load is transferred to the soil by friction FHWA (2005) [1]. Mini-piles construct in Europe at the beginning to retrofit the historic and the sensitive structures that had damage during World War II. As the development of geomechanics, a need for mini-piles was increased. Mini-pile technology is predicted to begin in Italy during the 1950 by Fernando Lizzi, and it was seen in the United States in the 1980 Bruce (1997) [2].

Since then, much collaboration and research has been done in the area of mini-pile design and construction. Today, mini-piles are effectively utilized in various scenarios including building underpinning, excavation stabilization, and
slope stability. A growing understanding of soil mechanics and the seismic effects on foundations has given rise to the use of mini-piles. Deteriorating and ageing buildings require foundation retrofitting to resist the continuing loads from the environment and structure. The versatility of mini-pile construction has allowed effective stabilization of many older structures Barron (2016) [3].

There are many studies have been lead to evaluate the performance of mini-pile in sandy soil under various types of loading. Different techniques for testing have been employed such as full-scale mini-pile load tests, 1g physical modeling, and the centrifuge modeling. Jeon & Kulhawy (2001) presents a full-scale field tests on 21 mini-piles with different diameters and lengths, eight mini-piles were installed in clay soils and thirteen in sandy soils. The results of the test indicate that the mini-pile load carrying capacity per pile volume can be higher than larger diameter drilled shafts for shaft depth to diameter ratio less than 100. This increase is about 1.5 to 2.5 for mini-piles installed in sandy soil [4].

Tsukada et al. (2006) evaluated the improvement in bearing capacity of spread footings reinforced with mini-piles in sandy soil has different densities by using small models. They found that the bearing capacity of the foundations increased in dense sand [5]. Kyung et al. (2016) presented an experimental testing program on a series of model load tests to investigate the vertical load-carrying behavior of mini-piles for various parametric such as installation angle, pile spacing, and foundation type applicable for mini-piles [6]. Tae (2017) investigated the characteristics of a mini-piled raft through model tests and a numerical analysis. The behavior of the mini-piled raft is evaluated for various conditions, such as soil type, pile length, and installation angle [7]. Sharma et al. (2019) studied the behavior of an experimental model for group mini-piles in sand. In this study three important parameters, length to diameter ratio (L/D), spacing of piles and relative density of sand were considered. Efficiency of the groups increases with the increase in L/D ratio at 30% and 50% relative density but it is opposite at 80% relative density [8]. Geotechnical centrifuge testing was conducted by Alnuaim et al (2015) in order to investigate the behavior of mini-piled raft foundations with different raft thickness in sandy soil and evaluate their performance characteristics [9].

Several authors have been presented results of numerical simulations for settlement reducing piles, Rose et al. (2013) [10] Investigated the performance of micro pile groups in clays using geotechnical centrifuge testing and numerical modeling. Alnuaim et al. (2016) presented a numerical investigation on the performance of micro piled raft foundation in sand. Calibrated and verified finite element model with centrifuge tests [11]. Zolfegharifar et al. (2015) Presents analyze the group function of mini-piles with different length and numbers on a two-layer soil bed with using the ABAQUS software and three dimensional modeling [12]. El Sharnouby (2018) investigated through three-dimensional finite-element analysis the axial compression behavior of reinforced helical pull-down micro piles and fiber-reinforced polymer. The model was calibrated using the results of full-scale load tests conducted by the authors [13].

A number of studies have been evaluated the piled raft foundation performance such as Poulos & Davis (1974) [14]), Randolph (1994) [15], Poulos (2001) [16], Tuna (2016) [17], and Ling-Yu Xu et al. [18] Mahboubi & Nazari-Mehr, (2010) evaluated the single and groups mini-pile performance in sandy soil under dynamic loading and using a finite element method to validate the results of centrifuge test [19]. In addition, several mini-pile tests were evaluated the lateral performance of mini-piles such as Richards & Rothbauer (2004) [20], Shahrou & Ata (2002) [21], Teerawut (2002) [22], and Kershaw & Luna (2018) a total of seventeen model micro piles were tested to investigate the effect of simultaneous axial and lateral loading on micro pile foundations, the results of the study indicated that the lateral deflection was only significantly affected by the introduction of a constant axial load for large lateral loads [23].

In addition to focus on the geotechnical aspect of the settlement reduction and how it is affected by the mini-piles, this study innovatively concerns with the structural aspect of how using mini-piles can reduce the bending moments in raft foundation.

2. Materials and Methods

Full-scale tests are the best approach to predict the behavior of mini-piles. To reduce the cost, time, and effort of the test, small-scale models are usually used in the laboratory instead of the full-scale tests. On the other hand, the scale-down models have great advantage as they permit a full control for the geotechnical properties of the soil and for the details of the model used in the tests. Boundary conditions and loading of the model can be specified, to explain in what technique that the loads are applied. In addition, small tank and quantity of the soil, small lengths of the mini-piles and raft, and small load are required to carry out the test compared with those requirements in prototype Wood (2004) [24].

In this study, the testing program consisted of the following tests: (1) one test on a single mini-pile; (2) one test on a raft with a thickness equivalent to 15 mm (3) three tests on five mini-piled group with different raft thicknesses of 8mm, 10mm and 15mm. All tests were performed on a dry sandy soil with a relative density, D_r, of 60%. All piles have a diameter of 6mm and a length of 178mm. Scale-down for all models has been determined based on Equation 1 and 2.

\[
\frac{E_mA_m}{E_pA_p} = n_l^2n_G
\] (1)
\[
\frac{E_m I_m}{E_p I_p} = n_t^4 n_G
\]

Where: \(E_p A_p\) is the axial rigidity for the prototype element of the mini-pile.
\(E_m A_m\) is the axial rigidity for the model element of the mini-pile.
\(E_p I_p\) is the flexural rigidity for the prototype element of the mini-pile.
\(E_m I_m\) is the flexural rigidity for the model element of the mini-pile.
\(n_t\) is scale factor for length \((n_t = 1/n)\), and \(n = 45\).
\(n_G\) is scale factor for shear stiffness, \((n_G = 1/n^\alpha)\). Experimental experience suggests that the exponent \(\alpha\) might be in the order of 0.5 for sands Wood (2004) [24].

In order to work with reasonable size, the model rafts and piles were fabricated using PVC with elastic modulus, \(E\), of 2900 MPa and Poisson ratio, \(\nu\), of 0.4 \((E=2900\ \text{MPa}, \nu=0.4)\), which has a modulus of elasticity smaller than that of the prototype material, concrete. Table 1 shows the models and the corresponding prototypes dimensions along with the appropriate scaling laws:

<table>
<thead>
<tr>
<th>Description</th>
<th>Scaling law</th>
<th>Prototype (m)</th>
<th>Model (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mini-pile Diameter</td>
<td>(E_p A_p = n^{(2+\alpha)})</td>
<td>0.20</td>
<td>6</td>
</tr>
<tr>
<td>Mini-pile Length</td>
<td>(1/n)</td>
<td>8.0</td>
<td>178</td>
</tr>
<tr>
<td>Raft Width</td>
<td>(1/n)</td>
<td>5.25</td>
<td>117</td>
</tr>
<tr>
<td>Raft Thickness</td>
<td>(E_p I_p = n^{(4+\alpha)})</td>
<td>0.6</td>
<td>15</td>
</tr>
<tr>
<td>Raft Thickness</td>
<td>(E_p I_p = n^{(4+\alpha)})</td>
<td>0.4</td>
<td>10</td>
</tr>
<tr>
<td>Raft Thickness</td>
<td>(E_p I_p = n^{(4+\alpha)})</td>
<td>0.3</td>
<td>8</td>
</tr>
</tbody>
</table>

3. Test Setup

All model tests were conducted using the setup shown in Figure 1, which consists of a loading frame, loading jack, loading cell, piled-raft, and soil tank. The vertical load was applied to the models by means of 10-ton hydraulic compression handle jack. During all the experimental tests, the loading rate is kept approximately constant at 3 mm/min. The applied load is measured using a “Sewha, Korea” load cell five-ton capacity. Two LVDT have been used for measuring displacements of the models. Four strain gauges were attached to the piles while three strain gauges were attached to the raft. All of them were connected to a data logger with ten channels to measure the strain, two channels to measure the load, and two channels to measure the displacement. The data logger in turn was connected to a computer through an interface device to read the strain value.
The adopted soil tank has interior dimensions of 0.4 m length, 0.4 m width, and 0.5 m height. These dimensions are proposed to be compatible with test supporting frame. To have no interference between walls and the soil, a clear distance of 142 mm was provided from the face of the raft to the interior face of the wall tank. For the same purpose, a distance of two-times pile length was provided from the raft to the bottom of container as shown in Figure 2.

![Figure 2. Soil tank with boundary condition](image)

4. Soil Properties

The soil used for the model tests is clean, oven-dried, uniform quartz (Kerbela) sand. The maximum and minimum dry unit weights of the sand were determined according to the ASTM (D4253) and ASTM (D4254) specifications, respectively, the specific gravity test is performed according to ASTM (D854), the grain size distribution is analyzed according to ASTM (D422) specifications and direct shear test according to the ASTM (D3080). Figure 3 shows the grain size distribution of the sand and Table 2 summarize the physical properties of the tested sand. The angle of internal friction is determined using the direct shear test, which was carried out for the sand. The height of the sand was divided into 4 layers of 100 mm by knowing the weight and the volume of the sand it was sure compacted to 60% relative density ($D_r$).

![Figure 3. Grain size distribution of the sand](image)
Table 2. Physical properties for the tested sand

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective size, $D_{10}$</td>
<td>0.27</td>
</tr>
<tr>
<td>Coefficient of uniformity, $C_u$</td>
<td>2.89</td>
</tr>
<tr>
<td>Coefficient of curvature, $C_c$</td>
<td>0.96</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.64</td>
</tr>
<tr>
<td>Maximum unit weight, $\gamma_{d_{max}}$</td>
<td>17.5 kN/m$^3$</td>
</tr>
<tr>
<td>Minimum unit weight, $\gamma_{d_{min}}$</td>
<td>14.3 kN/m$^3$</td>
</tr>
<tr>
<td>Test unit weight, $\gamma_{d_{test}}$</td>
<td>16.06 (kN/m$^3$)</td>
</tr>
<tr>
<td>Relative density, $D_r$</td>
<td>60%</td>
</tr>
<tr>
<td>Angle of friction $\phi$</td>
<td>40°</td>
</tr>
</tbody>
</table>

5. Strain Measurement

This experiment work adopts plastic strain gauges type GFLA-3-50-3LJC from TML, with the following characteristics: wire-type with a resistance of 120 Ω, a gauge factor of 2.09 +/- 1%, a gauge length of 3mm and width of 2.5mm.

Four strain gauges were attached to the PVC mini-piles, as indicated in Figure 4, to measure the strains at every load increment in the following locations:

- At the top and bottom of a corner mini-pile;
- At the top and bottom of the center mini-pile.

The strain gauge was covered with a layer of SB TABE size of 10mm width and 3mm thick to protect it from damage.

Figure 4. Strain gauges at corner and center piles with SB TABE

As indicated in Figures 5 to 6, three strain gauges were attached to the PVC raft as at the following locations to measure the strains:

- At an edge on the top surface;
- At the corresponding edge on the bottom surface;
- Near the center of the top surface.

Figure 5. Strain gauges installation at the top raft
6. Testing Procedure

The procedure followed in testing of the mini-piled raft models can be described in the following steps:

6.1. Building the Mini-piled Raft Foundation Model

PVC mini-piles have been grouped as indicated in Figure 7. Each pile has diameter of 6 mm and length of 178 mm. Notations adopted in the experimental work and in the subsequent discussion of the results have been summarized in Table 3 and Figure 7.

<table>
<thead>
<tr>
<th>Studied cases</th>
<th>Test Notations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single mini-pile</td>
<td>MP</td>
</tr>
<tr>
<td>Raft thickness 15mm</td>
<td>R_{15}</td>
</tr>
<tr>
<td>Raft thickness 15mm with five mini-piles</td>
<td>MPR_{15}</td>
</tr>
<tr>
<td>Raft thickness 10mm with five mini-piles</td>
<td>MPR_{10}</td>
</tr>
<tr>
<td>Raft thickness 8mm with five mini-piles</td>
<td>MPR_{8}</td>
</tr>
</tbody>
</table>

As indicated in Figure 8, the mini-piles were attached to the raft using a super glue to simulate a semi-fixed connection condition.
6.2. Preparation of Sand Deposit and Placing of Mini-Piled Raft Model

The sand was placed in the tank with four layers, each layer of 100 mm thick and has a compacted relative density, \(D_r\), of 60%. After each test, the tank should be emptied from the sand. During the sand filling process, the piled-raft model was placed at the center of the tank, then the sand was filled up to the bottom surface of the raft as presented in Figure 9.

6.3. Application of the Vertical Load

A concentrated vertical load is applied through a five-ton load cell at a constant loading rate. For each load increment, a sufficient time was given to full development of the corresponding strains and deformation. After each step, the load and the settlement were read the data logger through the corresponding channels. All gathered information have been saved in an Excel file. At end of the test, the sand was removed and the model was lifted.

7. Results and Discussion

7.1. Load Displacement Curves

Results for the load-displacement curves of the mini-piled raft foundation models are presented in Figures 10 to 14. Each figure contains two displacements from the attached two LVDTs. The average reading has been determined and presented also. Comparing between the two LVDT readings and their average value indicates that all foundations have uniform settlement during the loading process.

The load-displacement curves show that there is a 45% decrease in settlement for MPR15 comparing with R15, moreover there is no significant difference in settlement between MPR15, MPR10 and MPR8, where there is 1.5% difference between MPR15 and MPR10 and 7.8% between MPR15 and MPR8. This indicates that the considered different raft thicknesses have no significant effect on the geotechnical aspect of settlement reduction.
Figure 10. Load displacement curve for the raft R

Figure 11. Load displacement curve for MPR15

Figure 12. Load displacement curve for the MPR10

Figure 13. Load displacement curve for the MPR8
7.2. Bending Moment in the Rafts

The raft strain obtained from the test was used to calculate the moment M. Comparing the bending moments at the mid and edge of the mini-piled raft cases with those of the raft case indicates that the mini-piles significantly influence the structural behavior of the foundation.

Figure 15 presents the ratio between the bending moment at the mid of the mini-piled raft MPR15 to the corresponding bending moment at the raft R. It shows that the presence of the mini-piles decreases the bending moment ratio from 1 to 0.75 at the initial load and from 1 to 0.317 at the final load to indicate that there is a significant reduction, about 31.7%, in the moment at center of the mini-piled raft foundation compared with the corresponding moment of the control raft R. It is worthwhile to mention that the mini-piles have more significant role at the final loading stages comparing the initial loading stages.

Figure 16 presents the ratio between the bending moments at the mid of the mini-piled raft for different raft thicknesses to the bending moment of MPR15. This figure shows that the bending moment increases as the raft thickness decreases where it increases from 1 to 8.2, 4.9 in MPR10 and to 14.1, 10.8 in MPR8 at initial and final load respectively. These results indicate that the raft thickness has a significant effect on the moment distribution where a larger thickness leads to a more uniform distribution of moments that in turn reduce the maximum moment in the foundation.

Figure 14. Load displacement curve for the MP

Figure 15. Bending moment at mid raft for R and MPR15

Figure 16. Bending moment at mid rafts for MPR
Figure 17 presents the ratio between the bending moments at the edge of the mini-piled raft MPR15 to the corresponding bending moment at the raft R. It shows that the presence of the mini-piles decreases the bending moment ratio from 1 to 0.74 at the initial load and from 1 to 0.8 at the final load, in other words, there is 20% reduction in the edge moment due to using of the mini-piles with raft foundation has a large thickness.

Finally, Figure 18 presents the bending moments ratios at the edge of the mini-piled raft for different raft thicknesses. As indicated in this figure, the bending moment at the edge increases as the raft thickness decreases. It increases from 1 to 3.68, 3 in MPR10 and to 11.2, 9.7 in MPR8 at initial and final load respectively. This can be explained in terms of thickness role where a larger thickness leads to a more uniform distribution of the moment and in turn to a reduction in the maximum moment value.

7.3. Axial Load in Top and Bottom Mini-Piles

The strain of the mini-piles obtained from the test was used to calculate the axial load, \( P \), at the top and bottom of the mini-pile as indicated in Equation 3:

\[
P = \varepsilon E A
\]

Where \( A \) is the cross-section area of the mini-pile, \( \varepsilon \) is the axial strain measured from the strain gage, and \( E \) is the elastic modulus of PVC. Curves for the top and bottom axial load of the mini-piles at each load increment are presented in Figures 19 to 25. These figures illustrate that the axial load in the central mini-pile is higher than the axial load in corner mini-pile for all models. This trend of the results seems natural as the load is applied at the center of the raft and it acts directly on the central mini-pile. It is also observed that the axial load for the central and the corner mini-piles increases as the raft thickness decreases to indicate that mini-piles are more effective with thin models.
Figure 19. Load-depth curve for center mini-pile MPR15

Figure 20. Load-depth curve for corner mini-pile MPR15

Figure 21. Load-depth curve for center mini-pile MPR10

Figure 22. Load-depth curve for corner mini-pile MPR10
The friction for the mini-piles was calculated from Equation 4 and it has been presented in percentage form as indicated in Table 4 below. This table shows that most of the mini-pile axial loads are transferred to the underneath soil through friction and that the friction for the central and corner mini-piles increases as the raft thickness decreases.

\[ \text{friction} = P_{\text{top}} - P_{\text{bottom}} \]  

(4)

Table 4. Percentage of the friction to the total pile load at initial and final loading

<table>
<thead>
<tr>
<th>Mini-pile</th>
<th>Percentage of friction at the initial loading (%)</th>
<th>Percentage of friction at the final loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center mini-pile MPR15</td>
<td>73</td>
<td>69</td>
</tr>
<tr>
<td>Corner mini-pile MPR15</td>
<td>68</td>
<td>69</td>
</tr>
<tr>
<td>Center mini-pile MPR10</td>
<td>75</td>
<td>73</td>
</tr>
<tr>
<td>Corner mini-pile MPR10</td>
<td>71</td>
<td>72</td>
</tr>
<tr>
<td>Center mini-pile MPR8</td>
<td>76</td>
<td>74</td>
</tr>
<tr>
<td>Corner mini-pile MPR8</td>
<td>72</td>
<td>73</td>
</tr>
<tr>
<td>Single mini-pile</td>
<td>57</td>
<td>47</td>
</tr>
</tbody>
</table>
7.4. Load Carried by Mini-Piles or Raft

Comparison of load carried by the mini-pile and the raft are presented in curves of Figures 26 to 28. At the initial loading stage, most of the load is carried by the mini piles due to reduction of the contact between the raft and underneath soil. As the applied load increases, the proportion of the load carried by the piles gradually decreases. At load of 0.8 kN approximately, the load transferred by the mini-piles becomes almost constant. At final stage, the loads carried by the rafts for the MPR8, MPR10, and MPR15 foundations respectively were 83%, 85% and 86% of the total applied loads.

Figure 26. Percentage of load carried by mini-piles or raft curve for MPR15

Figure 27. Percentage of load carried by mini-piles or raft curve for MPR10

Figure 28. Percentage of load carried by mini-piles or raft curve for MPR8
8. Conclusions

Five tests were conducted to investigate the behavior of mini-piled raft foundations, with a varying raft thickness, in a dry sandy soil under a concentrated vertical load. The following conclusions may be pointed:

- There is a 45% decrease in settlement for 15mm mini-piled raft foundation comparing with the reference 15mm raft foundation. Moreover, there is no significant difference in settlement between 15mm mini-piled raft foundation comparing with the 10mm and 8mm thick mini-piled raft foundations.
- The bending moment decreases at the mid and edge of the 15mm mini-piled raft foundation comparing to those of the reference raft foundation. It has also been noted that the moments are inversely proportional to the thickness of the piled raft foundations.
- With respect to the mini-piles, it has been found that most of the pile axial loads are transferred to the underneath soil through friction and this friction increases as the raft thickness decreases.
- The loads carried by the rafts for the 8mm,10 mm,15 mm thick mini-piled raft foundations respectively were 83%, 85% and 86% of the total applied loads.

9. Funding

This work supported by the laboratories of Building Research Directorate, Ministry of Construction Housing and Municipalities, Baghdad, Iraq.

10. Conflict of Interest

The authors declare no conflict of interest.

11. References


Adoption of Prefabrication in Small Scale Construction Projects

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Received 07 September 2018; Accepted 03 April 2019

Abstract

The construction industry is facing numerous difficulties in managing construction waste, quality, environment, permanence, safety, and greater construction cost. Dynamic change is needed today to overcome new challenges in the construction industry. Adoption of prefabrication is one of the possible solutions to such problems. This paper explores the advantages in prefabrication adoption with its possible disadvantages (barriers) through the qualitative study. This paper is an addition to the existing literature of prefabrication specially for developing countries where the acceptance rate of new approaches is difficult. It covers private residential project and a public housing project. This study also aims to evaluate the current status of prefabrication adoption in small-scale construction projects. A set of the questionnaire is used to collect the data and Average Index (AI) method using SPSS has been used to analyze the results. Shorter construction time, Low site waste and better supervision are the main advantages. Higher initial construction cost and Strict & difficult design changes are the key disadvantages. It is analyzed that the conventional construction method is more frequently used when compared with prefabrication concept.

Keywords: Prefabrication; Waste Management; Small Scale Projects.

1. Introduction

Increasing awareness of environmental, social and economic issues in today’s building methods has allowed practitioners around the world to adopt practices that are considered more sustainable in the long term. In the construction industry, conventional on-site construction methods have long been criticized for their durability, low productivity, low level of safety and a large amount of waste [1, 2]. As an alternative to these problems, prefabrication can provide significant benefits, such as reduced time, low waste, improved quality, reduced environmental emissions, improved work environment, and reduced energy and water consumption [3, 4]. One of the main reasons for the discouragement of decision-makers to adopt prefabrication is that they have difficulty in finding the benefits that such an approach would add to a project [5]. In fact, prefabrication is not always the only solution available, and it is not always better than the on-site construction method because of the different characteristics of the project and the resources available. If not used properly, orders lag significantly behind the production, cost overruns and structural problems in the use of prefabrication. Deciding to use prefabrication based on confidentiality and personal preferences is not uncommon [6]. Pasquire and Connolly (2002) has shown that the decision to include prefabrication still relies heavily on subjective evidence, rather than hard data, as there are no formal measurement strategies [5].

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http://dx.doi.org/10.28991/cej-2019-03091314
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Prefabrication is widely regarded as a sustainable construction method with regard to its impact on the protection of the environment. An important aspect of this perspective is the influence of prefabrication on the reduction of construction waste and subsequent waste management activities, including waste categorization, recycling and disposal [7]. Recent studies reported that in order to cope up with the challenges in speed and quality in the construction industry to offset the shortage of houses for growing population in any country, the need of the day is prefabrication [8]. The use of prefabrication technology can contribute to waste reduction significantly. On condition that more detailed designs, waste reduction during construction could be achieved by avoiding unsuccessful works and unnecessary repetition of works [9]. Compared with the traditional cast-in-place method, it has been unable to meet the requirements of the construction industry and the development of the times. Because the prefabricated building has the advantages of fast speed, water saving, land saving, noise reduction, material saving and energy saving in installation [10]. Zhai (2017) explored the effect of operative hedging and develops the coordination mechanism towards a definite hedging problem in the prefabricated construction supply chain management [11]. Bon-Gang et al. (2018) reported that prefabrication can improve the workflow continuity, increase the efficiencies in the use of resources, minimize construction wastes, and reduce the number of on-site contractors as well as construction durations [12]. Many studies have focused on the technologies and reasoning behind off-site construction [13]. Prefabricated construction has attracted worldwide concern because of its significant role in the creation of sustainable urbanization [14]. Prefabrication is an innovative and cleaner approach that has restructured the production of the construction industry [12]. Fard et al. (2015) highlighted that prefabrication is also prone to occupational accidents so it is also important to evaluate it [15].

Prefabricated construction is becoming more common, improved in quality and has become available in a variety of costs. Many benefits are reported for this approach, including green approaches, financial savings, and flexibility in design, consistent quality, reduced site disruption, reduced construction time, and improved productivity. The results of Jaillon and Poon (2008) showed that the environmental, economic and social benefits of prefabrication were significant compared to conventional construction methods [9]. This implies that wider use of prefabrication techniques can contribute to sustainable construction in a close urban environment. In order to improve the overall quality and efficiency, it is necessary to increase the way the construction is carried out and revised. The key lies in innovation and blocking the many barriers that limit the sector’s enormous potential to create a sustainable built environment. Hence, it is essential to evaluate this panorama that would encourage the suitable discussion of the appropriateness of prefabrication and other construction methods. Thus, this paper is an initial step toward this serious problem. The study aims to identify advantages of prefabrication and barriers in the adoption of it. It also aims to investigate the current status of prefabrication adoption in the construction industry of Pakistan. This paper will provide a pre-requisite knowledge and scenario of prefabrication adoption in making small-scale building projects. The results of this study may lead to a broader research for prefabrication adoption in big and complex construction projects.

2. Sustainability Aspects of Prefabrication

Sustainability enables a holistic response to environmental, social crises and creates the necessary links between nature, culture, economy, politics and technology. Prefabricated elements provide environment-friendly, energy and cost-efficient solutions for the building [16]. Prefabricated modular structures are increasingly becoming popular [17, 18]. This is starting to lead customers to consider the effects of the sustainability of the construction, operation and maintenance of projects. Today’s World is striving to cope up with upcoming challenges including saving natural resources, enhanced use of recycled items, environmental degradation, the overall cost of construction item and so on. All of this can be achieved by enforcing existing sustainability theories and modifying the sustainability aspects. The result of this struggle, which is evident in the highly developed and still developing countries, is closely linked to the pressures of economic progress. The framework for sustainable infrastructure design should review the economic impact of new prefabricated and construction technologies.

3. Research Methodology

An extensive literature review has been made to explore the gape in the existing body of knowledge for prefabrication and its acceptance level in different countries followed by Pakistan. After identify a gape, a research method was designed to carry out this research work. In the later stage, pilot studies were conducted to seek stakeholder’s opinion for the prefabrication and its factors and finally a set of questionnaire was designed to collect data from the construction industry. It is observed that Average Index has been successfully used as a decision-making approach for such data set so the same approach is adopted for this study. The final ranks are based of this approach. The complete research methodology is shown in Figure 1.
4. Data Collection and Analysis

A detailed literature review has been made for factor identifications in this research. The identified factors were processed through a short pilot study. Expert’s opinion during pilot study is amended in final set of questionnaire which was send to numerous practitioners working in construction industry via hard mail and emails. The respondents were requested to share their experience in assess the adoption level of prefabrication, advantages and disadvantages of prefabrication in general and with specific reference to small scale residential projects at private side and government side. Finally, 159 questionnaires were considered for this research which was received during data collection period.

Average Index (AI) method has been successfully used for data analysis of such decision-making problems. Therefore, same is used for data analysis of this paper. Average Index is indexed as shown in Eq: 1

\[
\text{Average Index} = \frac{\sum_{i=1}^{5} a_i X_i}{\sum_{i=1}^{5} X_i}
\]

(1)

Where, \(a_i\) = Constant expressing the weight given to \(i\), \(X_i\) = variable expressing the frequency of the response for:

5. Results and Discussion

As discussed earlier, the respondents were requested to share their opinion based on their work experience in construction industry. The respondents were provided with a 4-point likert scale and requested to weight the factors which are advantageous for prefabrication in construction industry of small-scale residential building projects. Table 1 shows the rank of factors which are advantageous for prefabrication based on AI score.

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Advantages of Prefabrication</th>
<th>AI Score</th>
<th>Rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Shorten construction time</td>
<td>3.57</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>Low site waste</td>
<td>3.48</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>Better supervision</td>
<td>3.46</td>
<td>3</td>
</tr>
</tbody>
</table>
It is observed that shorten construction time and less construction site waste are ranked as first and second with an average mean value of 3.57 and 3.48 respectively. It indicates that while adopting prefabrication in construction, will cause the overall shorten project duration and due to the manufacturing of components at particular site or in factory it will result less construction site waste. Also, the better supervision, sustainable product and environmentally friendly are ranked as third and fourth followed by others as shown.

Other than the advantages in adopting prefabrication, the disadvantages on the applications of prefabrication are also investigated in this research. Similar analysis has been made for this phase of the research. Table 2 shows the responses on the disadvantage (hindrances) in applying prefabrication in building construction projects.

<table>
<thead>
<tr>
<th>Sr. No</th>
<th>Disadvantages of Prefabrication</th>
<th>All Score</th>
<th>Rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Higher initial construction cost</td>
<td>3.25</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>Strict &amp; difficult design changes</td>
<td>3.12</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>Time consuming in initial design</td>
<td>2.91</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>Leakage problem while joining prefabricated components</td>
<td>2.87</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>Lack of availability of prefabricated industries</td>
<td>2.85</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>Lack of skilled labour</td>
<td>2.85</td>
<td>5</td>
</tr>
<tr>
<td>7</td>
<td>Lack &amp; expensive equipment</td>
<td>2.83</td>
<td>6</td>
</tr>
<tr>
<td>8</td>
<td>Limited site space</td>
<td>2.78</td>
<td>7</td>
</tr>
<tr>
<td>9</td>
<td>Lack of materials used in prefabrication</td>
<td>2.75</td>
<td>8</td>
</tr>
<tr>
<td>10</td>
<td>Fewer demand by clients</td>
<td>2.67</td>
<td>9</td>
</tr>
<tr>
<td>11</td>
<td>Government legislations and Guidelines</td>
<td>2.66</td>
<td>10</td>
</tr>
<tr>
<td>12</td>
<td>Transport requirements and may limit its scope</td>
<td>2.62</td>
<td>11</td>
</tr>
<tr>
<td>13</td>
<td>Limited trained labour</td>
<td>2.62</td>
<td>11</td>
</tr>
<tr>
<td>14</td>
<td>Lack of experiences</td>
<td>2.55</td>
<td>12</td>
</tr>
<tr>
<td>15</td>
<td>Increased production volume is required to ensure affordability through prefabrication</td>
<td>2.45</td>
<td>13</td>
</tr>
<tr>
<td>16</td>
<td>New process and unfamiliarity of process</td>
<td>2.45</td>
<td>13</td>
</tr>
</tbody>
</table>

It is observed that higher initial cost and Strict & difficult design changes are ranked as first and second with an average mean value of 3.25 and 3.12 respectively. Since the prefabricated components are manufactured early at the stage and if in future it is required to change the design of the project then it will be inflexible and prove to be costly. In
addition to it, time consuming in initial design and leakage problems while joining prefabricated components stands at third and fourth factor followed by others as shown.

Finally, current status of the adoption of prefabrication has been assessed in this research. Comprehensive prefabrication method for different elements of projects is shown in Table 3.

<table>
<thead>
<tr>
<th>NO.</th>
<th>Building Element</th>
<th>Sub Elements</th>
<th>Private Residential Projects</th>
<th>Public Housing Projects</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Substructure</td>
<td>Foundation</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Basement</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>2</td>
<td>Drainage and underground work</td>
<td>Drainage</td>
<td>70%</td>
<td>80%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Piling</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>3</td>
<td>Structural frame works</td>
<td>Column</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Beam</td>
<td>15%</td>
<td>5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stairs</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slab</td>
<td>5%</td>
<td>0%</td>
</tr>
<tr>
<td>4</td>
<td>External works</td>
<td>Boundary Wall</td>
<td>30%</td>
<td>60%</td>
</tr>
<tr>
<td>5</td>
<td>Internal works</td>
<td>Partition Wall</td>
<td>15%</td>
<td>5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fall Ceiling</td>
<td>100%</td>
<td>100%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tiling</td>
<td>75%</td>
<td>80%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washroom</td>
<td>15%</td>
<td>10%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Kitchen</td>
<td>25%</td>
<td>10%</td>
</tr>
</tbody>
</table>

It is observed that private and public sector is widely using prefabrication for fall ceiling. They also use prefabrication significantly for drainage and tiling works. Both sectors use prefabrication for kitchen items, washroom fixtures, boundary walls and partition walls to some stage means they are adopting it for such items for building works. It is observed that both sectors started adopting this concept to some extent for elements like beams, columns and slabs. Whereas, it is also observed that still both sectors lack to use prefabrication concept for foundation, basement, piling and stairs, they are not accepting this idea as a better replacement for cast-in-situ elements as mentioned.

6. Conclusion

Prefabrication method for the construction industry provides a much more efficient atmosphere for productivity, eliminating the unnecessary distractions and interference typically encountered in conventional construction sites. It should be noted that prefabrication in most cases takes less than half the time compared to traditional construction. This is due to better planning, design, elimination of on-site problems and meteorological factors, subcontractor scheduling delays and faster manufacturing, as multiple parts can be built simultaneously. Prefabrication is a possible solution to the main causes of waste that arise in design and construction. Prefabrication also contributes to other benefits at the site, such as shorter construction time, better monitoring can be achieved with respect to the environment, improving quality and sustainability. Reducing total construction costs and better aesthetic prospects are also important advantages of prefabrication. Considering the results, it can be concluded that adoption of prefabrication is becoming a norm in building construction, though conventional method is still used in majority of construction industry but looking at scenario of construction in developed countries it seems that the use of prefabrication should likely to increase. With the continued popularity of prefabricated construction, it is likely to continue to grow in popularity. Customers who choose this option can benefit from a high-quality, faster, cost-effective and environmentally friendly construction method.

7. Acknowledgement

The authors are thankful to Mehran University of Engineering & Technology, Jamshoro, Pakistan for providing platform to conduct this research at master’s level. The authors also extend their gratitude to Prince Sultan University, Riyadh, Saudi Arabia for expertise role throughout this research study.

8. Conflicts of Interest

The authors declare no conflict of interest.
9. References


Comparison Mechanical Properties of Two Types of Light Weight Aggregate Concrete

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Received 18 March 2019; Accepted 18 May 2019

Abstract

This paper presents the behavior of concrete properties by replacing the conventional coarse aggregate used in the concrete mixture by two types of lightweight aggregate; Expanded Perlite Aggregate (EPA) and Volcanic Pumice (VP). To fulfill this aim; three laboratory tests were applied; density, compressive strength, and abrasion resistance, that conducted to extrapolate the range of the changes in the properties of concrete with existence those types of aggregate in the mixture. Also, the volumetric proportion adopted as a strategy for replacing the coarse aggregate by EPA or VP in the concrete mixture. Then, the volumetric proportion ranged from 10% to 50% with the variation step was 10%. Therefore, ten concrete mixtures are prepared and divided into two groups; each group contains five concrete mixes to represent the volumetric replacement (10-50)% of conventional coarse aggregate by EPA or VP. On the other hand, one extra mixture designed by using conventional aggregate (coarse and fine aggregate) without any inclusion of EPA or VP to be considered as a reference mixture. The obtained laboratory results of this study proved that the density, compressive strength, and abrasion resistance readings of concrete decreased at any volumetric proportion replacement of coarse aggregate by EPA or VP. The decrease in density and compressive strength of concrete readings amounted the peak level at 50% replacing of coarse aggregate by EPA, which were 38.19% and 77.37%, respectively than the reference mixture. Additionally, the compressive strength is an important factor affecting the abrasion resistance of concrete mixture, and loss of abrasion decreased as compressive strength increased.

Keywords: Lightweight Aggregate Concrete; Volumetric Replacement; Density; Compressive Strength; Abrasion Resistance.

1. Introduction

Lightweight Concrete (LWC) is a versatile material that has a great interest and large industrial demand in recent years in a wide range of construction projects, despite its known use dating back over 2000 years. The mineral admixtures, fibers and prolonged age of curing are the effective parameters to control the shrinkage cracking in the LWC [1]. As regards to the economic aspect, the using LWC in the floor slabs would reduce the total costs of the tall buildings through decreasing the foundation volume, the amount of steel reinforcement, and vertical members’ cross-sections that saves the used horizontal area [2].

The thermal conductivity of LWC is ranged from 0.2 to 1.0 W/m.K besides the oven dry density range from roughly 300 to not exceed 2000 kg/m³, with a cubic compressive strength about 1 to more than 60 MPa. These ranges could

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http://dx.doi.org/10.28991/cej-2019-03091315

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compare to those for normal weight concrete with 1.6 to 1.9 W/m.K, roughly 2100 to 2500 kg/m³, and 15 to greater than 100 MPa [3].

Alshihri et al. [4] pointed out that the LWC could employ for rapid evolution in construction applications of high rise buildings larger-sized and long-span concrete structures. According to Neville and Brooks [5], there are three kinds of LWC: the first kind through substitution the normal weight aggregate by lightweight aggregate of low specific gravity (lower than 2.6) and this kind of concrete is well known as lightweight aggregate concrete. The second kind inducing bubble voids within the concrete or mortar mass, the concrete is known as aerated, cellular, foamed, or gas concrete. Through eliminating the fine aggregate from the mix, this is considered as the last kind of LWC; and it is known as no-fines concrete. Li [6] concluded the structural lightweight concrete status achieved when the concrete has a compressive strength more than 17 MPa with a bulk density less than 1950 kg/m³. Furthermore, Vilches et al. [7] classified the LWC into three groups based on their use and physical properties (for structural use, for both structural/insulating purpose and for insulating).

Jedidi et al. [8] studied the influences of replacement sand by EPA on the compressive strength and thermo-physical properties of LWC at different ages. The researchers indicated that the compressive strength decreased when the perlite dosage increased, but on the other hand, the existence EPA improved the thermal resistance of concrete.

Yu et al. [9] designed an ultra-LWC that could enhance both thermal insulation and the load bearing element from lightweight and binder aggregates. They referred to that the excellent thermal characteristics achieved with a thermal conductivity of approximately 0.12 W/m.K; and at the age of 28 days, the compressive strength ranged from 10 to 12 N/mm². Through using the extrusion strategy by Lu et al. [10], the magnesium oxchloride cement with the expanded perlite/paraffin composite has a significant impact on developing the energy storage capacity in buildings. Depending on the study submitted by Różycka and Pichór [11], the use expanded perlite waste in autoclaved aerated concrete has a positive effect on the formation of calcium silicate hydrates. To increase the compressive strength of concrete, El Mir and Nehme [12] included the waste perlite powder in the self-compacting concrete mixtures as a filler material.

The natural lightweight aggregates such as pumice, diatomite, scoria, volcanic cinders, tuff, also artificial lightweight aggregates such as perlite, expanded clay, slate, shale, and vermiculite have employed as construction substances [13].

Kurt et al. [14] noticed that when the pumice proportion increased caused to increase the segregation tendency in fresh concrete. By using the pumice as lightweight aggregate, and different ratios of blast furnace slag and water/cement (cement and mineral additive); Kurt et al. [15] studied the mechanical and physical characteristics of self-compacting lightweight aggregate concrete. It observed that when the ratio of blast-furnace slag is constant and the increase in the amount of pumice aggregate; the unit weights decreased and the water absorption ratios increased. The lightweight concrete with pumice and tragacanth additive can be employed as dividing and coating material in the constructions due to its insulating features [16]. Wijatmiko et al. [17] pointed out that the use coated pumice in the concrete mixture caused to increase the flexural strength of beam by 2.58% as compared to uncoated ones. Numan et al. [18] used volcanic pumice as an alternative material to the total volume of crushed coarse sand in the concrete mixture. They deduced that the replacement volcanic pumice instead of fine aggregate exhibited less impact resistance than the replacement of volcanic pumice instead of coarse aggregate.

Numan [19] added a super-plasticizer and Glenium 51 as additives substances to the concrete mixture to rise from the compressive resistance of concrete. The treatment pumice aggregate by sodium hypochlorite solution has a significant impact on increasing the compressive strength of concrete [20]. Cemalgil and Onat [21] investigated the effect of using waste marble aggregate and waste demolition materials in the concrete mixture on the engineering properties of concrete. They found that when using demolition waste the abrasion resistance of concrete significantly decreased. Karataş et al. [22] detected that the increment dose of the lightweight aggregate (expanded perlite and pumice) would decrease the tensile strength in bending and compressive strengths of mortar specimens subjected to high temperatures. On the other hand, to minimize the impact waste on the environment, besides improving the mechanical properties and fire resistance of construction material; Yaseen et al. [23] reused plastic waste resulting from mineral water bottles as a different adding ratio to gypsum mixture. They disclosed that the addition 1% of plastic waste resulted in increasing of the compressive strength and modulus of rupture of gypsum by 18.75% and 26.32%, respectively; while adding 1.5% resulted in enhancing the fire resistance of gypsum.

Different studies were conducted to develop the structural action and sustainability of reinforced concrete members through using or inserting a green material on or throughout their, such as Kadhim et al. [24]. The scholars conducted a numerical analysis to evaluate the structural action of reinforced concrete by externally wrapping it with basalt fiber-reinforced polymer. They detected that the ultimate load was increased by 14.8% in spite of 20% corrosion in the flexural steel rebar under eight layers of basalt fiber-reinforced polymer composite.

Based on the presented literature survey and besides to search the best of the authors’ knowledge, it is found that the studies interesting to emulate the abrasion resistance of concrete with existence EPA and VP materials are indeed scarce. Also, for more highlight on the trend and behavior of the relationship between the compressive strength and abrasion
resistance of concrete under different volumetric ratios of replacing traditional coarse aggregate by these materials. Consequently, the experimental program is presented to clarify the target relation and emulate the influences of existence EPA and VP in the concrete mixture on the physical-structural response of concrete.

2. Research Methodology

In this paper, the experimental work accomplished to emulate the influence of addition two types of lightweight material; EPA or VP on the changing properties of concrete. The volumetric ratio strategy was adopted to add these materials instead of coarse aggregate in the mixture. The volumetric ratio was 10, 20, 30, 40, and 50%, respectively.

The essential objective of this paper is to specify the variance between properties of concrete with the presence EPA once and other times with presence VP in the concrete mixture. Moreover, the properties of concrete that contained EPA or VP as an additive material compared to the properties of concrete without any addition to specifying the real amount of changing of the physical-structural demeanor of concrete. To achieve this objective; eleven laboratory concrete mixtures prepared and undergone to (density, compressive strength, and abrasion resistance) test. It is worth noting that in the lab of Building and Construction Technology Engineering/Northern Technical University in Mosul/Iraq, the mixing of concrete, preparation of specimens, as well as, material (the main ingredient of concrete) and specimen tests done.

3. Materials’ Specifications

The specifications and characteristics of the materials used in this work are clarified in the following subsection:

3.1. Cement

The Ordinary Portland Cement (OPC-Type1) that produced from Badoosh Cement Factory used. The physical, mechanical, and chemical characteristics of the cement used, besides to the cement limitations based on Iraqi Organization for Standards and Specifications (IQS, No. 5,1984) [25] are given in Table 1 and 2, respectively. From these tables, it observed the characteristics of cement used within the range of the Iraqi Organization for Standards and Specifications (IQS, No. 5, 1984) [25].

<table>
<thead>
<tr>
<th>Table 1. Physical and mechanical characteristics of cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristics</td>
</tr>
<tr>
<td>-----------------</td>
</tr>
<tr>
<td>Specific surface area, Blaine method, cm$^2$/gm.</td>
</tr>
<tr>
<td>Standard consistency, %</td>
</tr>
<tr>
<td>Setting time, Vicat’s method</td>
</tr>
<tr>
<td>Initial setting, min.</td>
</tr>
<tr>
<td>Final setting, min.</td>
</tr>
<tr>
<td>Fineness on sieve No. 170, %</td>
</tr>
<tr>
<td>Compressive strength of 50 mm cubic mortar samples, N/mm$^2$</td>
</tr>
<tr>
<td>3 days</td>
</tr>
<tr>
<td>7 days</td>
</tr>
<tr>
<td>Tensile strength, N/mm$^2</td>
</tr>
<tr>
<td>3 days</td>
</tr>
<tr>
<td>7 days</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 2. Chemical analysis of cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compound composition (%) cement used</td>
</tr>
<tr>
<td>-------------------------------------</td>
</tr>
<tr>
<td>$\text{Al}_2\text{O}_3$</td>
</tr>
<tr>
<td>$\text{SiO}_2$</td>
</tr>
<tr>
<td>$\text{Fe}_2\text{O}_3$</td>
</tr>
<tr>
<td>$\text{CaO}$</td>
</tr>
<tr>
<td>$\text{SO}_2$</td>
</tr>
<tr>
<td>$\text{MgO}$</td>
</tr>
<tr>
<td>$\text{C}_3\text{S}$</td>
</tr>
<tr>
<td>$\text{C}_2\text{S}$</td>
</tr>
<tr>
<td>$\text{C}_3\text{A}$</td>
</tr>
<tr>
<td>$\text{C}_4\text{AF}$</td>
</tr>
</tbody>
</table>
3.2. Coarse Aggregate

The washed coarse aggregate used that gathered from Mosul city. It was rounded and with a maximum size of 12.5 mm. The findings of the sieve analysis of coarse aggregate used illustrated in Table 3. As shown in Table 3, the accumulated percentage passing of coarse aggregate used conformed to British Standard (BS: 882, 1992) [26], within the Limit 5-14 mm (Fine) specification. On the other hand, the specific gravity and absorption capacity of the coarse aggregate used was 2.66 and 0.4%, respectively.

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>Accumulated percentage retained (%)</th>
<th>Accumulated percentage passing (%)</th>
<th>BS: 882,1992 specification [26]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>5-14 mm</td>
<td>5-20 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(Fine)</td>
<td>(Medium)</td>
</tr>
<tr>
<td>14</td>
<td>0.0</td>
<td>100</td>
<td>90-100</td>
</tr>
<tr>
<td>10</td>
<td>15.70</td>
<td>84.30</td>
<td>50-85</td>
</tr>
<tr>
<td>4.75</td>
<td>91.40</td>
<td>8.60</td>
<td>0-10</td>
</tr>
<tr>
<td>2.36</td>
<td>100</td>
<td>0.0</td>
<td>----</td>
</tr>
</tbody>
</table>

3.3. Fine Aggregate

The rounded natural sand with a maximum size 4.75 mm collected from Khazer region in Mosul/Iraq used in all concrete admixtures as a fine aggregate. Also, the sieve analysis conducted on the sand used. The outcomes of sieve analysis for the sand used depicted in Table 4. From this Table, it disclosed that the accumulated percentage passing of the sand used was within the range of BS: 882, 1992 [26] under the fine limitations. The specific gravity, absorption capacity, and material finer than sieve No. 200 of the sand used were 2.68, 2.88%, and 0.8%, respectively.

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>Accumulated percentage retained (%)</th>
<th>Accumulated percentage passing (%)</th>
<th>BS:882,1992 specification [26]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Limits</td>
<td>(Fine)</td>
</tr>
<tr>
<td>4.75</td>
<td>0.0</td>
<td>89–100</td>
<td>60–100</td>
</tr>
<tr>
<td>2.36</td>
<td>11.0</td>
<td>60–100</td>
<td>80–100</td>
</tr>
<tr>
<td>1.18</td>
<td>25.5</td>
<td>60–100</td>
<td>80–100</td>
</tr>
<tr>
<td>0.60</td>
<td>44.5</td>
<td>15–100</td>
<td>55–100</td>
</tr>
<tr>
<td>0.30</td>
<td>78.5</td>
<td>5–70</td>
<td>5–70</td>
</tr>
<tr>
<td>0.015</td>
<td>96.5</td>
<td>0–15</td>
<td>----</td>
</tr>
</tbody>
</table>

3.4. Expanded Perlite Aggregate (EPA)

Expanded perlite is a glassy volcanic rock, honey-comb like or bubble-like, white or grayish-white granules as shown in Figure 1. It has some advantages such as good performance in low temperature, light weight, incombustibility, better chemical stability, easy application, and corrosion resistance. In construction, the perlite considered good thermal insulation and also sound-absorbing products. The thermal conductivity of perlite within the range of 0.06 to 0.17 W/m. K under high temperatures and of 85 kg/m³ density [27].

![Figure 1. Expanded Perlite used](image)

3.5. Volcanic pumice (VP)

From Hatay region/Turkey, the Volcanic Pumice (VP) collected that used as an alternative material to gravel in concrete mixtures. It is an aggregate with light gray colored coarse aggregate, as shown in Figure 2. Due to the VP used has a low density (835 kg/m³); therefore it can float on the water. The VP consists of different chemical compounds, the percentage of main these compounds depicted in Table 5 based on Saad [28].
Silica Fume

The silica fume (SikaFume-HR) that manufactured by Sika Near East s.a.l Beirut-Lebanon with fineness 0.1μm is used. The technical data of SikaFume-HR collected and illustrated in Table 6 based on information from product factory. The aim of using silica fume is to improve the strength of concrete, which may be ascribed to the pozzolanic reaction Ca(OH)$_2$ crystals located in the transition zone. As a result, it will enhance the bond between the cement particles and aggregate surface.

<table>
<thead>
<tr>
<th>Property</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composition</td>
<td>A latently hydraulic blend of active ingredient</td>
</tr>
<tr>
<td>Appearance</td>
<td>Grey powder</td>
</tr>
<tr>
<td>Dry bulk density</td>
<td>0.05 – 0.1 kg</td>
</tr>
<tr>
<td>Dosage</td>
<td>2–10% by weight of cement</td>
</tr>
</tbody>
</table>

Chemical Admixtures

Super-plasticizer (Sika ViscoCrete-SF 18) used as high range water, minimizing admixture and viscosity modifying agent. The used potion was 1% by weight of cement. The characteristics of super-plasticizer used, as provided by the manufacturer, are illustrated in Table 7 [29].

<table>
<thead>
<tr>
<th>Property</th>
<th>Sika ViscoCrete-SF 18</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chemical base</td>
<td>Modified poly carboxylates based polymer</td>
</tr>
<tr>
<td>Appearance / color</td>
<td>Light brownish liquid</td>
</tr>
<tr>
<td>pH value</td>
<td>3 – 7</td>
</tr>
<tr>
<td>Density</td>
<td>1.1 g/cm$^3$ ± 0.02, (at + 20°C)</td>
</tr>
<tr>
<td>Dosage</td>
<td>1.0 - 2.0% by weight of cement</td>
</tr>
</tbody>
</table>
3.8. Water

In this paper, all concrete mixtures and curing process of samples done by using ordinary water from the tap without any additives and treatment.

4. Method of Mixing and Preparation of Specimens

The drum mixer with a capacity of 0.071 m$^3$ was used to pour all batches of concrete. Each batch was used to cast the six iron cube specimens of (100×100×100) mm, and six iron cylinder specimens of (150×75) mm to perform the compressive strength, and abrasion resistance tests. All molds are cleaned and oiled their interior surface before using them.

Table 8 shows all details the mix design proportions of the reference concrete mixture (C1). The reference concrete mixture was prepared depending on recommendations of the American Concrete Institute (ACI 211.1-91, 1991) [30] to attain a compressive strength of concrete not less than 28 MPa.

To obtain the structural response of concrete under replacement conventional coarse aggregate by EPA or VP materials; ten concrete mixtures prepared besides the reference mixture. The volumetric ratio method was adopted to illustrate the replacement technique of coarse aggregate by these materials (the volumetric ratio ranged from 10% to 50%). The C2, C3, C4, C5, and C6 represent the concrete mixtures under replacement technique of conventional coarse aggregate by EPA with the variable step of 10%, while C7, C8, C9, C10, and C11 represent the concrete mixtures under replacement technique of conventional coarse aggregate by the VP with also variable step of 10%.

It is worth to mention that the current replacement technique adopted by removing a ratio of coarse aggregate and substituted by an equivalent ratio of EPA or VP only, while the other constituent proportions of the reference concrete mixture were kept constant. Table 9 depicts the concrete mix proportions of all mixtures designed. As a more clarified the current replacement technique; some calculations inserted at the end of Table 9. All specimens immersed in curing tank at 23°C and relative humidity more than 90% till testing at the age of 28 days. Figure 3a and 3b depicts some specimens after the expiry of the curing period.

Table 8. Mix Proportion of Concrete Constituents

<table>
<thead>
<tr>
<th>Constituents</th>
<th>Water (kg/m³)</th>
<th>Cement (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Gravel (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weights (%)</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>1176.5</td>
</tr>
<tr>
<td>Percentages</td>
<td>0.5</td>
<td>1</td>
<td>1.864</td>
<td>3.361</td>
</tr>
</tbody>
</table>

Table 9. Compositions of Mixes

<table>
<thead>
<tr>
<th>Mixture designation</th>
<th>Water (kg/m³)</th>
<th>Cement (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Gravel (kg/m³)</th>
<th>Perlite (%)</th>
<th>Pumice (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix C1</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>1176.5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Mix C2</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>1058.85*</td>
<td>10</td>
<td>3.85**</td>
</tr>
<tr>
<td>Mix C3</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>941.20</td>
<td>20</td>
<td>7.00</td>
</tr>
<tr>
<td>Mix C4</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>823.55</td>
<td>30</td>
<td>11.55</td>
</tr>
<tr>
<td>Mix C5</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>705.90</td>
<td>40</td>
<td>15.40</td>
</tr>
<tr>
<td>Mix C6</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>588.25</td>
<td>50</td>
<td>19.25</td>
</tr>
<tr>
<td>Mix C7</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>1058.85*</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Mix C8</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>941.20</td>
<td>0</td>
<td>20</td>
</tr>
<tr>
<td>Mix C9</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>823.55</td>
<td>0</td>
<td>30</td>
</tr>
<tr>
<td>Mix C10</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>705.90</td>
<td>0</td>
<td>40</td>
</tr>
<tr>
<td>Mix C11</td>
<td>175</td>
<td>350</td>
<td>652.5</td>
<td>588.25</td>
<td>0</td>
<td>50</td>
</tr>
</tbody>
</table>

* Gravel in C2 = (1176.5 – (0.1 × 1176.5)) = 1058.85
** Perlite in C2 = (0.1 × 1176.5)/2.6 × 0.085 = 3.85
*** Perlite in C7 = (0.1 × 1176.5)/2.6 × 0.835 = 37.78
Unit weight of used aggregate = 2.6
Unit weight of used Pumice = 0.835
Unit weight of used Perlite = 0.085
5. Tests of Specimens

The laboratory tests are divided into two stages depending on the status of concrete. With fresh concrete status, the densities for all concrete mixtures are tested and considered the first stage of tests. On the other hand, the second stage of tests began when the concrete achieved hardened status (at the age of 28 days). At this stage, both the compressive strength and abrasion resistance tests for all prepared specimens accomplished.

5.1. Fresh Density Test

According to the American Society for Testing and Materials (ASTM C138/C138M-01a, 2001) [31] specifications; all fresh concrete mixtures density examined. The density value is calculated through determining the net weight of freshly mixed concrete and divided by the volume of concrete produced from a mixture at the moment of casting.

5.2. Hardened Concrete Tests

5.2.1. Compressive Strength Test

When the cube specimens reached the age of 28 days, they subjected to the compressive strength testing. This test carried out by using the Uniaxial Testing Machine (UTM) of capacity 2000 kN, at the rate of loading of 0.5 MPa/s and it comforted to specifications of BS 1881: Part 116, 1983 [32]. Figure 4 illustrates the UTM and cube specimen which is ready to test.

5.2.2. Abrasion Resistance Test

To give a real allusion of the relative abrasion resistance of concrete based on testing of currently fabricated specimens, the abrasion resistance test conducted on the cylindrical specimens. The age of these specimens was 28 days. The test apparatus composes of rotating cutter and drill press. All the steps that conducted to fulfill the abrasion resistance test conformed to the recommendations of ASTM C944/C944M-12, 2012 [33]. These steps summarized as follows:
• The mass of the specimen was determined to the nearest 0.1 grams.

• The rotating cutter device mounted in the drill press shaft.

• When the electric motor is turned on, the cutter lowered slowly until being in contact with the surface of the concrete specimen. Then, the abrasion process started of normal load (roughly 10 kg) on the surface specimen for two continuous minutes. At the end of two minutes; the specimen removed from the rotating cutter, cleaned the surfaces from debris using a soft brush, and weighed specimen. These actions repeated with the same time interval (two minutes) with the minimum test schedule shall involve three 2-min periods carried on three separate areas of the concrete surfaces. The test stopped when the weight of the specimen stabilizes or 1.0 mm (depth of wear); whichever happened first. Figure 5 shows the conducted abrasion resistance test by using the rotating cutter drill press and the balance scale.

![Figure 5. The specimen under the rotating cutter drill press](image)

6. Results and Discussion

6.1. Fresh Properties Results

Under all designed composition systems of concrete mixtures; the findings of the unit weight test are clearly illustrated in Table 10 and Figure 6, respectively. In Table 10 NWC denotes to Normal Weight Concrete, while LWAC denotes to Light Weight Aggregate Concrete.

<table>
<thead>
<tr>
<th>Mixture designation</th>
<th>EPA%</th>
<th>VP%</th>
<th>Density (kg/m³)</th>
<th>Type of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix C1</td>
<td>0</td>
<td>0</td>
<td>2435</td>
<td>NWC</td>
</tr>
<tr>
<td>Mix C2</td>
<td>10</td>
<td>0</td>
<td>2100</td>
<td>NWC</td>
</tr>
<tr>
<td>Mix C3</td>
<td>20</td>
<td>0</td>
<td>1865</td>
<td>LWAC</td>
</tr>
<tr>
<td>Mix C4</td>
<td>30</td>
<td>0</td>
<td>1695</td>
<td>LWAC</td>
</tr>
<tr>
<td>Mix C5</td>
<td>40</td>
<td>0</td>
<td>1565</td>
<td>LWAC</td>
</tr>
<tr>
<td>Mix C6</td>
<td>50</td>
<td>0</td>
<td>1505</td>
<td>LWAC</td>
</tr>
<tr>
<td>Mix C7</td>
<td>0</td>
<td>10</td>
<td>2145</td>
<td>NWC</td>
</tr>
<tr>
<td>Mix C8</td>
<td>0</td>
<td>20</td>
<td>1960</td>
<td>LWAC</td>
</tr>
<tr>
<td>Mix C9</td>
<td>0</td>
<td>30</td>
<td>1925</td>
<td>LWAC</td>
</tr>
<tr>
<td>Mix C10</td>
<td>0</td>
<td>40</td>
<td>1895</td>
<td>LWAC</td>
</tr>
<tr>
<td>Mix C11</td>
<td>0</td>
<td>50</td>
<td>1845</td>
<td>LWAC</td>
</tr>
</tbody>
</table>
As shown in Table 10 and Figure 6, the value of density in C2 and C7 is not much affected as compared to the reference mixture, where the decrease in C2 and C7 density was 13.75% and 11.91%, respectively than C1. On the other hand, the density value of C2 and C7 stills within the range of NWC (2000-2500) kg/m$^3$. Therefore, the impact of replacing 10% coarse aggregate by (EPA or VP material) in the concrete mixture is considered somewhat limited.

As it is evident in Table 10 and Figure 6, the kind of concrete automatically turned to normal to lightweight (the density of lightweight concrete less than 2000 kg/m$^3$) when increasing the percentage dosage of EPA or VP more than 10%. However, the incremental dose of EPA in the concrete mixture by (20, 30, 40, and 50) % in (C3, C4, C5, and C6) accompanied in decreasing the concrete density of (23.41, 30.39, 35.73, 38.19) % compared to the reference mixture. Additionally, the concrete density decreased as (19.51, 20.94, 22.18, and 24.23%) when transforming from the reference mixture to C7, C8, C9, C10, and C11, respectively. The decrease in the density of concrete when replacing coarse aggregate by EPA or VP materials are considered logically due to these materials have the lowest density values than the conventional coarse aggregate.

Another note recorded on the results in Table 10 and Figure 6 that is the replacement of coarse aggregate by EPA in the concrete mixture was more impact on the reducing density than to the replacement VP under the same level of volumetric ratio proposed. This because of the lower density of EPA as compared with the VP density.

### 6.2. Compressive Strength Test Results

At 28 days age of cubic specimens for all designed mix proportions of concrete, the compressive strength test conducted. The outcomes of this test are illustrated in Table 11 and Figure 7, respectively.

<table>
<thead>
<tr>
<th>Mixture designation</th>
<th>EPA %</th>
<th>VP %</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix C1</td>
<td>0</td>
<td>0</td>
<td>28.50</td>
</tr>
<tr>
<td>Mix C2</td>
<td>10</td>
<td>0</td>
<td>21.85</td>
</tr>
<tr>
<td>Mix C3</td>
<td>20</td>
<td>0</td>
<td>16.50</td>
</tr>
<tr>
<td>Mix C4</td>
<td>30</td>
<td>0</td>
<td>12.70</td>
</tr>
<tr>
<td>Mix C5</td>
<td>40</td>
<td>0</td>
<td>9.00</td>
</tr>
<tr>
<td>Mix C6</td>
<td>50</td>
<td>0</td>
<td>6.45</td>
</tr>
<tr>
<td>Mix C7</td>
<td>0</td>
<td>10</td>
<td>25.50</td>
</tr>
<tr>
<td>Mix C8</td>
<td>0</td>
<td>20</td>
<td>22.95</td>
</tr>
<tr>
<td>Mix C9</td>
<td>0</td>
<td>30</td>
<td>20.85</td>
</tr>
<tr>
<td>Mix C10</td>
<td>0</td>
<td>40</td>
<td>19.35</td>
</tr>
<tr>
<td>Mix C11</td>
<td>0</td>
<td>50</td>
<td>17.85</td>
</tr>
</tbody>
</table>
From the results in Table 11 and Figure 7, the maximum value of compressive strength recorded in C1 (without any percentage of replacement the conventional coarse aggregate by EPA or VP material) as compared to the other mixtures. The value of compressive strength in C1 is decreased by (23.33, 42.11, 55.43, 68.42, and 77.37)% when approached to C2, C3, C4, C5, and C6, respectively. Furthermore, the value of compressive strength of C1 decreased by (10.53, 19.47, 26.84, 32.11, and 37.37)% when approached to C7, C8, C9, C10, and C11, respectively. The inclusion EPA or VP material within the concrete mixture working to reduce the level of bonding between constituents of the concrete and increasing in the porosity level of the specimen simultaneously. For these reasons interpreted why the compressive strength value reduced in C2, C3, C4, C5, C6, C7, C8, C9, C10, and C11 as compared to the C1.

A closer look at the percentage results that are mentioned above in this section, it is found that the lowest percentage of decreasing in compressive strength of concrete occurred between C1 and C7, which was 10.53%. This result can aid the engineers to capture the compressive strength of concrete not much affected when the dosage of VP was 10% in the mix, and hence can be used mixture designed in C7 in some concrete applications with acceptable compressive strength.

The findings in Table 11 and Figure 7 demonstrate that the influence of adding EPA as an alternative substance to coarse aggregate was more effective from decreasing the compressive strength of concrete than the VP. This attributable is weak mechanical properties (especially compressive resistance) of the EPA material compared to VP material.

### 6.3. Abrasion Test Results

All the results gathered from the abrasion test in the current work inserted in Table 12, Figure 8, and Figure 9, respectively.

<table>
<thead>
<tr>
<th>Table 12. Abrasion test results for each mixture designation</th>
<th>Depth of wear (mm)</th>
<th>Abraded materials (gm.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix C1</td>
<td>0.355</td>
<td>14.80</td>
</tr>
<tr>
<td>Mix C2</td>
<td>0.44</td>
<td>18.15</td>
</tr>
<tr>
<td>Mix C3</td>
<td>0.505</td>
<td>21.50</td>
</tr>
<tr>
<td>Mix C4</td>
<td>0.56</td>
<td>25.00</td>
</tr>
<tr>
<td>Mix C5</td>
<td>0.625</td>
<td>28.50</td>
</tr>
<tr>
<td>Mix C6</td>
<td>0.68</td>
<td>32.00</td>
</tr>
<tr>
<td>Mix C7</td>
<td>0.38</td>
<td>15.55</td>
</tr>
<tr>
<td>Mix C8</td>
<td>0.423</td>
<td>17.37</td>
</tr>
<tr>
<td>Mix C9</td>
<td>0.465</td>
<td>19.12</td>
</tr>
<tr>
<td>Mix C10</td>
<td>0.501</td>
<td>20.60</td>
</tr>
<tr>
<td>Mix C11</td>
<td>0.542</td>
<td>22.34</td>
</tr>
</tbody>
</table>
As clearly shown in Table 12, Figure 8, and Figure 9 that the lowest depth of wear and abraded materials values recorded on the reference mixture, which was 0.335 mm, and 14.8 gm., respectively. The increment of replacement conventional coarse aggregate by EPA in C2, C3, C4, C5, C6 accompanied by the increasing in depth of wear by (23.94, 42.25, 57.75, 76.06, 91.54)% and also increase the abraded materials by (22.64, 45.27, 68.92, 92.57, 116.22)% as compared with C1. Additionally, the increment of VP dosage as replacement material instead of coarse aggregate in concrete mix as (10, 20, 30, 40, and 50)% caused to increase the depth of wear by (13.43, 26.27, 38.81, 49.55, 61.79)% and also increase the abraded materials by (4.73, 17.36, 29.19, 93.19, 50.95)% as compared with C1. The reason for increasing the depth of wear and abraded materials weights in the specimens is due to its contained either EPA or VP because of the low bonding achieved between the constituents of concrete and increased the porous for those specimens.

At the same volumetric proportion of replacement, the specimens with VP exhibited more strength to abrasion than those contained EPA as shown in Table 12, Figure 8, and Figure 9. These back that the total porosity of VP is relatively lower than the EPA, as well as, the compressive resistance of EPA is lower than the VP. The same reasons explain why the less abrasion resistance recorded at C6 that contain the highest volumetric proportion of EPA replacement to the total volume of conventional aggregate. Furthermore, the tendency of decreasing both compressive and abrasion resistance of specimens with increasing the EPA dosage in the concrete mixture was significant as compared to increase VP dosage.

Through the comparison of the results in Tables 11 and 12, it found that the inverse correlation achieved between compressive strength and abrasion resistance of concrete with any percentage dose of EPA or VP in the mixture.

For more visualization about the steps that carried out in this work, Figure 10 illustrates these steps as a flowchart.
7. Conclusions

Based on the present experimental outcomes in this article; the main conclusions can be summarized as follows:

- The increment of EPA dosage in the concrete mixture from 10 to 50% with a variation step was 10% contributes to the decrease the concrete density from 13.75 to 38.19% as compared with reference mixture. With the same variation step of increment; but using VP material, the range of decreasing the concrete density of 11.91 to 24.23% as compared to the reference mixture. Accordingly, the presence EPA in the concrete mix was a higher impact to decrease concrete density than VP.

- Under the maximum percentage of volumetric replacement coarse aggregate by VP material (50%), the decrease in the compressive strength of C11 did not exceed 37.37%, while under only 20% of volumetric replacement coarse aggregate by EPA (C3) the compressive strength decreased by 42.11% compared to reference mixture. Therefore, the influence of adding a VP as an alternative substance to coarse aggregate was less effective in the decrease compressive strength of concrete than the EPA.

- The trend of increasing both of depth of wear and abraded materials value was more with existence EPA within the concrete mixture than VP. Therefore, the specimen contains VP exhibited more resistance to abrasion than the
EPA. The highest depth of wear and abraded materials values achieved in C6 (with the highest percentage dosage of EPA) and was 0.68 mm, 32 gm, respectively.

- A closer look at the findings obtained from this work illustrates that the compressive strength is an important parameter affecting abrasion strength of the concrete mixture. Loss of abrasion decreased as the compressive strength increased.

8. Acknowledgements

The authors thank and gratitude to Al-Mustansiriyah University, which provides all facilities and necessary scientific support (reading and borrowing the available scientific references in its libraries and use of the internet unit, etc.) to complete this paper.

9. Conflicts of Interest

The authors declare no conflict of interest.

10. References


Analysis and Study on Crack Characteristics of Highway Tunnel Lining

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Received 15 December 2018; Accepted 10 April 2019

Abstract

Lining cracks are one of the most common diseases in highway tunnels, and the existence of lining cracks directly affects the overall stability and durability of tunnels, which has an important impact on the safe operation of tunnels, and it is necessary to analyze and study the characteristics of tunnel lining cracks. Combining with the detection data of multiple highway tunnels in the field, the different types of tunnel cracks are divided, and the classification numerical statistics method is used to obtain that the number and length of annular cracks in highway tunnel cracks are significantly higher than those of the other two kinds of cracks, and the longitudinal cracks in tunnel crack cracking degree are greater than the circumferential cracks and the inclined cracks. The influence degree of cracks on the safety of tunnel structure longitudinal cracks are relatively the largest, the inclined cracks are second only to longitudinal cracks, and the influence of cyclic cracks is relatively small. It provides reference for tunnel engineering design, construction, operation management and comprehensive improvement work.

Keywords: Highway Tunnel; Lining Crack; Crack Cause; Crack Law.

1. Introduction

With the rapid development of expressway and more and more highway tunnel construction, tunnel diseases have become an important problem in tunnel engineering at present, and tunnel lining cracks, as a common disease phenomenon, will have adverse effects on the stress, waterproofing, appearance and so on of tunnel structures. In order to ensure the durability of the tunnel in operation and the safety and comfort during the driving process, it is an important problem to be solved to study the tunnel crack characteristics.

Guoquan Li, et al. (2015) The safety analysis of the tunnel with cracking of lining is carried out by using Shuguang Analysis Software, and the different cracking depth of two-storey lining cracks is put forward to have a great influence on the safety coefficient of the structure [1]. Sulei Zhang, et al. (2015) based on the equilibrium strip resin model of the sharp point mutation model, the stability criterion of lining crack is established, and a model of lining crack diagnosis based on field monitoring data is constructed [2]. Haiqiang Wang, et al. (2016) put forward the comprehensive judgment method, the integration of each single index evaluation system, so that the safety evaluation results of belt fracture lining more comprehensive, objective and accurate [3]. Zhijie Wang, et al. (2017) the regulation effect is verified by numerical simulation method, and the distribution law and difference of disease in different position and longitudinal mileage of cross section are obtained [4]. Jie Huang, et al. (2017) combined with geological reports, using the method of finite

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http://dx.doi.org/10.28991/cej-2019-03091316

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element numerical simulation, it is concluded that the structure is easy to produce longitudinal structural stress cracks under the action of deformation pressure [5]. Xujuan Wu, et al. (2017) Design Image Acquisition System image acquisition of Common Highway tunnel lining part, practice proves that this method is feasible, not only improves the capacity of highway tunnel during detection, but also solves the problem of tunnel crack lining detection [6]. Jin Feng, et al. (2017) using the K-s test method, the characteristic distribution law of highway tunnel lining cracks is analyzed statistically from the aspect of crack length and width two, and the crack length and width are in accordance with the logarithmic normal distribution characteristics [7]. Jianchao Li, et al (2018) a tunnel lining crack detection system based on image recognition technology is proposed [8].

Tunnel lining is the main building of the project which bears the formation pressure and the deformation and collapse of the upper surrounding rock, and the fracture tunnel lining fracture caused by the tunnel lining structure destroys the stability of the tunnel structure, reduces the safety and reliability of the lining structure, affects the normal use of the tunnel, and even endangers the driving safety [9]. The analysis and study of the characteristics of tunnel cracks play an important role in the normal use and maintenance of tunnels. Taking the highway tunnel detection of lining cracks in 13 provinces of Hangzhou city as the research object, the general law of Highway Tunnel cracks is analyzed by using the method of classification numerical statistics. The research focuses on the law of fracture distribution of different kinds of lining and its influence on the safety of tunnel structure.

2. Causes of Lining Cracks in Highway Tunnel

There are many forms of tunnel lining cracks, and the causes of the cracks are also varied. However, the most direct reason for the cracks in tunnel lining is that the tunnel lining structure is subjected to uneven force or excessive force [10]. Tunnel lining is the main building of engineering which bears formation pressure and prevents the deformation and collapse of surrounding rock. The magnitude of formation pressure mainly depends on the engineering geology and hydrogeological conditions and the physical and mechanical characteristics of the surrounding rock, and at the same time, it also depends on the construction method. Whether the lining is timely or not is related to the quality of the project. Due to deformation pressure, loosening pressure, uneven distribution of strata along tunnel longitudinal and mechanical behavior, temperature and shrinkage stress, swelling or frost heaving pressure of surrounding rock, corrosive medium, artificial factors in construction, The cyclic load of the running vehicle will cause cracks in the lining structure. To sum up, the main reasons [11-13] leading to cracks in highway tunnel lining are plastic ground pressure, partial pressure, expansive ground pressure, cavity behind the lining, insufficient lining thickness, and construction of “first arch after wall” method. Improper handling of surrounding rock (including fault break zone, construction collapse, etc.), formation disturbance (proximity) Construction, mining surrounding mining), no inverted arch and other reasons.

3. Harm of Lining Crack in Highway Tunnel

Tunnel lining cracking is the main form of tunnel disease which destroys the stability of tunnel structure reduces the reliability of tunnel structure safety affects the normal use of tunnel and even endangers pedestrian and personal safety. The main hazards of tunnel lining cracks are:

- Reducing the bearing capacity of tunnel lining structure to surrounding rock.
- The cracks of lining are easy to leak, which can easily cause corrosion of steel bars and facilities in the tunnel, mud of the pavement bed, frost damage of lining in cold area, and affect the durability of the tunnel.
- The cracks caused by excessive deformation make the tunnel clearance smaller and affect the safe passage of vehicles.
- Under the operation condition, the operation cost of the cracking lining is increased when the construction and operation interfere with each other.

4. Classification of Lining Cracks in Highway Tunnel

Tunnel crack detector, detecting cracks picture Figure 1 Classification according to the strike relationship between cracks and tunnel axis [14]:

- Longitudinal cracks: tunnel lining longitudinal cracks parallel to the tunnel axis, in the arch, side wall will occur, its development may lead to the tunnel arch, side wall crack or even cause tunnel collapse.
- Circumferential cracks: tunnel lining circumferential cracks vertical tunnel axis, mostly occurred in construction joints, settlement joints, or occurred in the hole, bad geological zone and complete rock and strata junction.
- Inclined cracks: tunnel lining slant cracks and tunnel longitudinal axis of 45, around the angle, in the arch, side wall will occur.
5. Research Methodology

Tunnel lining is the main building of the project which bears the formation pressure and the deformation and collapse of the upper surrounding rock, and the fracture tunnel lining fracture caused by the tunnel lining structure destroys the stability of the tunnel structure, reduces the safety and reliability of the lining structure, affects the normal use of the tunnel, and even endangers the driving safety. The analysis and study of the characteristics of tunnel cracks play an important role in the normal use and maintenance of tunnels. Taking the highway tunnel detection of lining cracks in 13 provinces of Hangzhou city as the research object, the general law of Highway Tunnel cracks is analyzed by using the method of classification numerical statistics. The research focuses on the law of fracture distribution of different kinds of lining and its influence on the safety of tunnel structure.

5.1. Statistical Analysis of Crack Characteristics

A total of 2683 tunnel lining cracks were detected in 13 highway tunnels in Hangzhou. If classified according to crack strike, there are 912 longitudinal cracks, 349 inclined cracks and 1422 circumferential cracks. The ratio of each type of cracks is shown in Figure 2 and Figure 3.

Figure 1. Tunnel detection machine detects tunnel crack pictures

Figure 2. Distribution scale diagram of different types of cracks

Figure 3. Proportional Chart of the total length of cracks in different types of cracks
The types of cracks (length and width of cracks) of the arch and side wall are counted, and the results are shown in Table 1.

### Table 1. Results of statistical characteristics of various types of cracks

<table>
<thead>
<tr>
<th>Statistical characteristics</th>
<th>Crack length (m)</th>
<th>Crack width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Longitudinal crack</td>
<td>Inclined crack</td>
</tr>
<tr>
<td>Mean μ</td>
<td>4.36</td>
<td>2.62</td>
</tr>
<tr>
<td>Standard deviation σ</td>
<td>3.18</td>
<td>2.07</td>
</tr>
<tr>
<td>Maximum (Max)</td>
<td>34</td>
<td>14</td>
</tr>
<tr>
<td>Minimum (Min)</td>
<td>0.5</td>
<td>0.32</td>
</tr>
</tbody>
</table>

As can be seen from Figure 2, Figure 3 and Table 1:

1. In the total number of cracks in tunnel, the number of circumferential cracks is the most, accounting for 53% of the total number of cracks, the number of longitudinal cracks is the second, which accounts for 34% of the total number of cracks, the quantity of inclined crack is the least, accounting for 13% of the total number of cracks.

2. Among the three kinds of cracks, the length of circumferential crack occupies 49% of the total length of crack, and the proportion is still the largest. The length of longitudinal crack is second only to that of circumferential crack, accounting for 41% of the total length of crack, and the length of inclined crack is the least, accounting for 10% of the total length of crack.

3. Compared with the length and width of a single crack, the three kinds of cracks have a general law of longitudinal crack > circumferential crack > inclined crack.

### 5.2. Influence of Different Types of Cracks on Tunnel Structure Safety

Combined with engineering experience and analysis of the causes of crack formation, although the number and total length of circumferential cracks are more than those of the other two kinds of cracks, the circumferential cracks are generally caused by longitudinal uneven confining pressure and improper treatment of construction joints. It has little effect on the safety of tunnel structure. Although the number and total length of longitudinal cracks are slightly less than those of annular cracks, the degree of cracking of single longitudinal cracks is greater than that of circumferential cracks, inclined cracks, and longitudinal cracks are generally formed by bias, swelling ground pressure, and cavities behind the lining. The displacement and stress of surrounding rock caused by the change of stress, found in the actual engineering, the longitudinal cracks mostly occurred in the arch line and the arch roof. The development of longitudinal cracks may cause the tunnel to fall off arch, break the side wall and even cause the tunnel collapse, which has the greatest influence on the safety of tunnel structure. Although the inclined cracks are the least in number, the shortest in length and the lightest in the degree of cracking, the inclined cracks are generally caused by landslide, strike of rock strata, joints, etc., and the causes are relatively complex. Its influence on tunnel structure safety is second only to longitudinal crack.

### 6. Conclusions

This paper introduces the main causes of highway tunnel cracks and the classification of highway tunnel cracks. Combined with the data of detecting lining cracks in 13 tunnels in Hangzhou, different types of cracks are displayed directly by chart by using the method of classified numerical statistics, and the following general laws of cracks in highway tunnels are analyzed:

- In the total number of cracks in tunnel, the number of circumferential cracks is the most, accounting for 53% of the total number of cracks, the number of longitudinal cracks is the second, which accounts for 34% of the total number of cracks, the quantity of inclined cracks is the least, accounting for 13% of the total number of cracks.

- Among the three kinds of cracks, the length of circumferential crack occupies 49% of the total length of crack, and the proportion is still the largest. The length of longitudinal crack is second only to that of circumferential crack, accounting for 41% of the total length of crack, and the length of inclined crack is the least, accounting for 10% of the total length of crack.

- Compared with the length and width of the single crack, the three kinds of cracks have the general law of longitudinal crack > circumferential crack > inclined crack.

- The longitudinal crack has the biggest influence on the tunnel structure safety, the inclined crack is second only to the longitudinal crack, and the circumferential crack has little effect on the tunnel structure safety.
7. Funding and Acknowledgements

This work is sponsored by the Fund of Institute of Highway Science, Ministry of Transport (201733) and the Youth Fund of Taiyuan University of Science and Technology (20153014) which are gratefully acknowledged.

8. Conflict of Interest

The authors declare no conflict of interest.

9. References


Green Envelop Impact on Reducing Air Temperature and Enhancing Outdoor Thermal Comfort in Arid Climates

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Received 24 March 2019; Accepted 18 May 2019

Abstract

Today Urban Cities faces energy and environmental challenges due to increased population, higher urbanization. The building sector has a big responsibility as it acquires high consumption rates in global energy and environmental scenarios. It is thought that the built environment in Egypt is responsible for 26% of the total overall national energy consumption, 62% of the total electricity consumption and around 70% of resultant CO2 emissions. The increased use of electrical appliances causes Urban Heat Island effect (UHI), which affect major urban centres. Adding green elements to any urban area is proved to be an effective strategy with numerous benefits to enhance the city's ecosystem, also mitigate the urban heat island measures. In this research Green roofs/walls can regulate outdoor air temperature by 10°C and improve outdoor thermal comfort by 2 Predicted Mean Value (PMV) values. The modelling of green strategy models can take into consideration design developments in areas with hot and dry climatic zones. The properties of green walls can directly affect the results of thermal comfort as leafs absorbs, reflects and transmits solar radiation, and increases the evapotranspiration.

Keywords: Green Walls; Retrofitting; Green Building; Green Roof; Air Temperature Reduction; Thermal Comfort Reduction; City Scale.

1. Introduction

The built environment is the largest part of the physical and economic human-made capital [1], where the construction sector itself constitutes a major part of the gross national product (GNP) [2] and accounts for 40% of the world's resource and energy use [3]. In Egypt, the sector is a particularly significant contributor to the economy (averaging at 25% of the GNP) [4] and is also a key user of energy. It is thought that the built environment in Egypt is responsible for 26% of the total overall national energy consumption, 62% of the total electricity consumption and around 70% of resultant CO2 emissions [5]. It is therefore important that the sector be considered a key target of energy consumption policies.

Adding green elements to any urban area is proved to be an effective strategy with numerous benefits to enhance the city's ecosystem, also mitigate the urban heat island measures. Many researches over the past decade investigated the impact of greenery when added to cities; especially trees, studies shown how effective a tree can be in reducing air temperature compared to air conditioning units. Greenery have proved to improve thermal comfort at the local scale [6].
7, 8 and 9). Since walls represent a high portion of the exposed urbanized area, green walls have gained widespread attention for their accountable impact on reducing energy consumption, also for their impact on modifying urban climate. It's proved that whenever vegetation, are installed in high- or medium-rise buildings, there is a direct impact on energy use and urban heat island mitigation potential [10-12].

Green roofs give a few advantages at both building and city level. Coming up next are the most usually seen at urban scale: mitigation of urban heat island impact [13]; decline in storm water overflow [14]; upgrade of biodiversity in densely urban regions [15]; cleaning of air and water spillover [16]. At Building scale, green walls diminish the reasonable heat flux because of the cooling impact [17] along these lines diminishing the heating and cooling demand of a building [18], and improving human thermal comfort. This impact may change contingent upon the atmosphere conditions, and the dimension of protection extraordinarily in instances of structure retrofitting. The greater part of these numerous advantages are connected to the cooling impact because of the evapotranspiration procedure (ET) that humidifies the outside encompassing air, decreases the surface temperature of the walls when irrigated, and mitigates the urban heat island [19].

Ideas for new urban areas should introduce biodiversity into the urban condition. Safeguarding biodiversity despite urbanization, territory discontinuity, ecological debasement and environmental change is presumably one of the best difficulties within recent memory. Tomorrow’s urban areas should offer new types of green space, yet in addition to moderate the hotter urban atmosphere and the urban heat island impact. Green urbanism is an all-encompassing idea for tomorrow's in addition to energy urban areas that depends on the use of energy, water, green spaces, materials and adaptability. Its long term objectives are zero emissions, zero waste and the evasion of energy/water/material wastage.

In light of the challenges facing the city identified previously, this study proposes an experimental investigation of green envelop thermal effects in the context of climate change and sustainable building design. Based on a case study in Egypt, it addresses the following research objectives. It explores the impacts of green envelop (green wall) on building microclimate and examines the effects of underlying environmental factors such as soil thickness on green-wall thermal performance.

The research hypothesis states that a vegetated green element can enhance the microclimate in a very powerful way that enhances the ecosystem health and present comfort to microclimate around a building. The research will present solutions for urban densifications and its impact on ecosystem health. The methods outlined in this research could be useful for urban planning and building design.

2. Review of Green Envelop

In 2009, it was found that in every one of the cities around the world, the surrounding temperature could be diminished just by upgrading the greeneries, especially advancing the idea of urban greening as opposed to urban ranger service on the grounds that the last possessed more ground space, which was completely rare in thickly populated urban communities [20, 21]. The most widely recognized urban greening innovations are vertical greener systems and green rooftops. Tragically, the vertical greener systems is at its nucleation stage in these cities. The creators specified different advantages of joining vertical greener systems; the primary being the warm advantages accomplished for the external wall texture, measured as far as temperature lessening of roughly 11.6 °C the peak value attained summer months. Broad investigations performed in 1990 demonstrated that the vertical greener system contributed more to the smaller scale atmosphere of the building. On the off chance that the whole building texture (walls and rooftops) including the open yards were hung with green façades, it absolutely created a cool small scale climatic condition, in this manner contributing more to the micro-climate. Greeneries reduce the outside surface temperature of building walls through transpiration cooling and give incredible shading.

In 2013, a descriptive review was published to discuss the significance of green rooftops and the contrast between green rooftops and regular rooftops. The outcomes uncovered that a green rooftop has a critical part in adapting the urban condition by reducing the urban heat island (UHI) impact [22]. A green rooftop built up in urban zones with both sub-tropical and tropical islands, demonstrate that it can decrease the expansion of outside temperature by around 42% and the increase of indoor temperature by 8% amid the day time. Amid night, it can keep up 17% of the temperature in the outside condition, balancing out the temperature change. It was discovered that by substituting man-made building materials with vegetation, a normal temperature contrast of 20 °C existed between the most and the minimum vegetated areas checking the urban heat island in four regions of New York City.

3. Case Study

Cairo, a mega city of dense urban morphology, Cairo experiences serious deficiency of ground-level green spaces. The green-space deficiency has corrupted the nature of the urban condition and related ecosystem services [23]. Urban Cairo has a huge potential to embrace the green layer innovation. There has long been a poor awareness among decision-makers and the public of the potential benefits arising from green-roof or green wall installation. Cairo is a high-density
city situated in the middle of Egypt (30.04° N, 31.23° E) with an average altitude of 74m. Outdoor comfort investigation is during the hottest day 2\textsuperscript{nd} of August [24] and is calculated using PMV, and air temperature. Cairo experiences a hot-arid climate with average temperature around 35 °C and humidity of 56%. The study is conducted on Neighborhood scale and street scale. The present study aims to investigate the effect of green infrastructure on reducing air temperature and outdoor thermal comfort in a typical urban neighborhood in Cairo. El Mohandseen “The Engineers”, is a region in Giza, Egypt. During the 1970s, the populace expanded drastically, and the once rich estates neighborhood transformed into swarmed flat squares. Over the most recent 10 years, population growth and city migration have led to uncontrolled and enormous urban growth. Leasing 5-story buildings towards 15-story towers, this has led to the formation of large and growing compact city morphology, a legalized urban morphology of unimaginable density and compactness. This resulted to a deterioration of the natural environment and the living conditions of an ascending number of urban dwellers. Figure 1, show the case study map.

Figure 1. Highlighted region of Mohandseen (focus study area)

4. Research Methodology

Quantitative research is set for Neighborhood and street scale. The author depended on simulation software tools as they are the adequate tools available. Envimet were chosen for the study. All simulations occur in Greater Cairo and Delta climatic zone. Simulation sequence goes as follows; first a full city block base case will be modeled into Envimet. After that green walls will be added to compare the effects of adding vegetation on air temperature and outdoor thermal comfort (PMV). This is followed by a street block investigation for more clarifications on the impact of greenery impact on buildings in terms of temperature reduction and thermal comfort alteration (Table 1).

<table>
<thead>
<tr>
<th>Variable 1: Building Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable 2: Building Setback</td>
</tr>
<tr>
<td><strong>Objective</strong>: Outdoor Temperature &amp; PMV</td>
</tr>
</tbody>
</table>

(BH from 5 to 15F/ BS from 3 m) (BH: building Height, BS: Building Setback, F: no of Floors)

4.1. Description of ENVI-met Software Tool

Microclimate recreations are performed utilizing ENVI-met 4.0, a three-dimensional computational liquid elements non-hydrostatic S.V.A.T. (soil, vegetation, air, and exchange) display. This program models the surface-plant-air associations in urban conditions, has been widely approved and utilized as of late [25, 26]. The primary information parameters of ENVI-met recreations incorporate climate conditions, starting soil wetness and temperature profiles, structures and physical properties of urban surfaces, and plants. ENVI-met permits to compute the air temperature, water vapour weight, and relative moistness, wind speed.

In ENVI-met, Vegetation is demonstrated for its evapotranspiration procedures, shadow, and drag impacts. ENVI-met does figuring concerning both shortwave and long-wave radiation motions regarding shading, reflection, and re-radiation from structures and vegetation. The temperatures of the ground and building surfaces are at long last computed
from an energy balance of differential conditions understood utilizing the limited distinction strategy.

ENVI-met simulation ought to normally be done for something like 6 hours, yet a 24 hour time span is more common. In this exploration, the model was mimicked from 10am of August second to 6pm of August second, utilizing the neighbourhood climate station information revealed in Table 2 as beginning limit conditions.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Input value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Start date</td>
<td>August 2nd</td>
</tr>
<tr>
<td>Start time</td>
<td>10:00 am</td>
</tr>
<tr>
<td>Simulation time</td>
<td>8 hours</td>
</tr>
<tr>
<td>Wind direction</td>
<td>North West (345°)</td>
</tr>
<tr>
<td>Wind speed (10 m)</td>
<td>3 m/s</td>
</tr>
<tr>
<td>Specific humidity (2500 m)</td>
<td>7.0 g/kgda</td>
</tr>
<tr>
<td>Relative humidity (2 m)</td>
<td>60 %</td>
</tr>
</tbody>
</table>

### 5. Software validation

In order to validate the software, a comparison between the measured and predicted air temperature were conducted between 10:00 a.m. and 6:00 p.m. on 2nd of august 2018. Air temperature measured at the Centre of the case study at a height of 1.5 m above the ground in a residential area of Mohandseen, Egypt, using a global temperature thermometer for outdoor use (measurement accuracy 0.1 °C). The weather data of that day were used as the inputs of ENVI-met. The relative errors of measurements were less than 10% at most time points, which may have been caused by the errors existing in weather data. The field experiments validated that the accuracy of ENVI-met can meet the requirements of this study (Figure 2).

### 6. Simulation Input Data for the City Block

The first step of the simulation process is the creation of an area input file. For the creation of the Mohandseen area input file a grid size of x=4m, y=4m, z=4m were used. The total number of grid cells along the x axis were 600, the y axis 600 cells and the z axis 90 grid cells. The area is mixed of 15 floor, 10 floors and 5 floors residential buildings. Setback between buildings varies, but in most cases 8 meters back to back and 6 meters side to side. The second step necessary to run the model was the creation of the configuration file. The simulation started on the 2nd of august at 10:00 AM. The total simulation time is 8 hours. The initial temperature used for the simulation is 28°C and reached 41°C. The wind speed was 3 m/s. The wind direction is 345° (North West). The relative humidity used as 56%. The model ran at the 250x250x30 version. The simulation will investigate the impact of extensive and intensive green walls on air temperature and thermal comfort (PMV).

#### 6.1. Air Temperature Results for City Block

The output files from the ENVI-met simulation model were analysed and visualized through maps with the use of the LEONARDO 3.75 software. The air temperature at 2 meters was visualized for both cases at eight different hours of the same day. The initialization of the simulation is at 10 AM. Case A (without greenery), as seen in Figure 3, have average air temperature of 30°C mostly in the middle; with high solar radiation at noon, interacting with concrete, bricks and asphalt. The minimum recorded air temperature is 28°C and the maximum is 33°C. As for Case B (with greenery) shown in Figure 4, the green influences the air temperature around the close surroundings. Green and vegetation can
influence the surrounding air temperature strongly. The green area with the concrete paved block in the middle that are situated on the west side of it seems to cool the close surrounding area at a small level. Even though the building blocks the wind flow, at this time of the day. The area is shaded by the building and thus the air temperatures are lower. The warmest areas are observed around the buildings on the top right corner and bottom area of the map with average temperature 23°C. The temperature difference when compared with case A at 12:00 PM is approximately 8°C. Greenery have a huge impact on outdoor air temperature in arid climates. Figure 5 is a comparison between case A & B through the day. Air temperature reduction from installing extensive green wall with thickness 10 cm was better than the intensive green wall with 30 cm thickness. It is common to provide comfort with thicker medium offered, but, it seems that thinner soil don’t store or traps sun rays, unlike the thicker soil. The reason behind these outcomes is the climatic zone in which the experiment occurs and simply this phenomenon is called Thermal mass (Figure 6).

Figure 3. Air temperature of Case A at 12:00 PM

Figure 4. Air temperature of Case B at 12:00 PM
6.2. Thermal Comfort Results for City Block

ENVI-met was selected to evaluate the outdoor thermal comfort. Biomet was used to calculate the PMV values at noon. Simulation for both cases A and B is measured at 2 meters height from street surface. The PMV values ranges between 2 to 3.4 for case A; characterized as HOT. The PMV values for case B range from 0.46 to 2.13 at 12 PM; characterized as COOL. For Case B; most of the area has PMV values are between 1 and 1.57 with the majority of the block having good comfort conditions. The lowest values are observed in front at the north and west and in the middle. After this value the “coolest” values are noticed at the green walls surrounding the buildings. Difference between PMV values for both cases A & B is considered impactful. Around 2 PMV value reduction; that provides better pedestrian comfort in the City block. These details can be seen in the following map, Figures 6 and 7. The best thermal comfort sensation is noticed in the areas with natural surface materials. Figure 8 is a full day difference between case A and B.

Figure 5. Air temperature difference between case A and B

Figure 6. PMV for case A at 12 PM
7. Input Data for the Street Block

On the street scale, attention to climate around buildings is targeted. Base case C (without greenery) and case D (with greenery). A single street was considered for simulation, a 3D model is shown in Figure 9. The same meteorology settings inherited; with the initial conditions of 3 m/s for the wind velocity and 345 deg. for the wind course. The simple forcing for the air temperature and the relative humidity are used along one day time span, with a base air temperature of 28 °C at 10 am and a greatest estimation of 41 °C at 6 pm. The relative humidity was 56%. The total simulation time was 8 hours. The displayed zone has the accompanying measurements: 100 × 100 m². The model territory has been rendered with network measure 150 cells along the x hub, 150 cells along the y pivot and 22 cells along the z hub. The extent of a lattice cell was: dx=2 m, dy=2 m and dz=3 m. The model was turned off 0° according to the area of the structures to the fundamental North direction.
7.1. Air Temperature Results for Street Block

The distribution of air temperatures for the two cases C & D were plotted at noon, with 1.5 m above the ground. In Figures 11 and 12, Base case C recorded a maximum air temperature of 33 °C; mainly above the asphalt streets around the buildings. In between buildings the temperature was around 30 °C; a 3 °C reduction than the street due to shading. Figure 10 is a section elevation representing the air temperature. As it is show higher temperature is close to the asphalt street surface, reaches 33 °C; as you go higher the temperature starts alter and reduced to 30 °C.

Figure 11. Air temperature map Case C at 1.5 m above the ground; 12 pm
The distribution of air temperatures at noon for Case D (with green walls) is shown in Figure 13 reveals that we have obtained an obvious decrease in temperature with the air temperature from Case C. All buildings in Case D are surrounded with a green wall. As an outcome, they have a tendency to retain a noteworthy extent of the heat. Vegetation captures radiation and produces shading that adds to diminishing the urban heat discharge. The nearness of vegetated dividers in Case D restrains air cooling because of the air dissemination produced among vegetation and structures.

The air temperature in Case C is reliably higher than the air temperature in Case D. Higher temperatures in Case C happen on account of the thick centralization of materials like black-top and cement in structures. These materials ingest warm amid the day and discharge it gradually around evening time. It is the opposite for Case D which development materials absorbs heat gradually and mirror the radiation at day time. As indicated by numerous analysts, UHI for the most part achieves its highest intensity after sunset, when the urban surfaces have adequately warmed-up. Consequently, the urban region with elevated structures displays temperatures a few degrees higher than the surroundings region around evening time demonstrating the impact of UHI.

7.2. Thermal Comfort Results for Street Block

Figures 13 and 14 present the Predicted Mean Vote (PMV) results at noon for the two cases C & D. The findings of case C, as shown in Figure 13 indicate that the Predicted Mean Vote ranges from 2.49 to 3.48, this indexed as hot. In Case D the (PMV) is about 1.9 – 3.3 meaning that some places especially around buildings it is possible to obtain cooler areas than in case C. There is a difference in 1 PMV value between the two cases. This is due to implementing green walls around buildings; which enhances the outdoor climate, reduces air temperature and provide a better condition for pedestrian in arid climates.
8. Results Analysis

The temperatures of the urban surroundings are normally controlled by albedo of the materials that assimilate and discharge diverse amount of Latent heat. Subsequently, green walls is a decent alternative to supplant those surfaces with vegetation. Vegetation oversees the temperature examples of a microclimatic territory through transpiration the water, change the wind speed, and give shading on the heat permeable surfaces and adjusting the heat exchange among urban surface. Green dividers plays out a steady heat sink with evaporative cooling process, and decrease the retaining
measure of radiative vitality when contrasted with a solid surface. This low sun-based retaining normal for the green dividers diminishes the surface air temperature and lessens the warmth transition. What's more, green walls can give numerous natural advantages to the site other than temperature relief.

9. Conclusion

The green envelope on walls, proved the possibility of reducing the external heat of the residential neighbourhood by a large proportion throughout the 2nd of August, which is the hottest day. The air temperature of a city block in Mohandseen was calculated. This city block consists of three different building heights (5, 10 and 15 floors). The simulation was carried out from 10 am to 6 pm for the existing situation without adding a green layer and once again when a green wall was added. The results showed a temperature reduction by an average of 10 °C throughout the day when implanting the walls. This is followed by outdoor thermal comfort investigation, measured in PMV index. The results accounted for a difference of 2 PMV values (from HOT to COOL) for the whole city block. The properties of green walls can directly affect the results of thermal comfort as leaves absorbs, reflects and transmits solar radiation, and increases the evapotranspiration. These results are significant and impact the urban heat island in the city of Cairo.

10. Conflicts of Interest

The authors declare no conflict of interest.

11. References


Prioritizing the Main Elements of Quality Costs in Design-Build Mass-Housing Projects

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Received 06 January 2019; Accepted 12 April 2019

Abstract
Reducing the cost of quality in mass-housing projects can reduce the overall cost and can also result in increasing profitability or the possibility of getting more projects due to the lower price offered in the tenders. The first step to reduce the cost of quality is to identify different elements, determine their impact on the final product quality and then prioritize them. In this study, questionnaires and structured interviews with experienced construction professionals were employed to identify and prioritize the fundamental elements using the P-A-F (prevention, evaluation, and failure) method, one of the most well-known methods for categorizing quality costs. The results indicate a high impact of preventive activities and the low impact of external failure activities on final product quality. According to the results, the use of experienced specialists and skilled workers is more effective than in-service training of inexperienced forces. Corrective actions of non-conformities and design improvements have a significant impact on final product quality. The new approach to COQ elements ranking, used in this research, can help decision-makers to prioritize the most effective activities in construction projects to increase final quality with an optimum quality cost.

Keywords: Cost of Quality; Construction Projects; Mass-housing; Quality Management; PAF Method.

1. Introduction

In recent decades, the increasing need housing in developing countries resulted in mass-housing or complexes projects managed government's authorities. For example, several multi-million housing plans in Iran have been designed and implemented in the form of mass-housing construction projects within large cities suburbs [1]. In such circumstances, it is essential for construction companies to reduce the cost of housing without losing the expected quality to attend in bidding competition. COQ (Cost of Quality) identification not only provides the opportunity to quantify and record costs, but also makes it possible to identify poor quality products and thus reduce costs by better and more appropriate use of resources and facilities. In addition, COQ can identify the areas where the total cost of quality can be optimized to increase quality level [2] and also can be useful as an overall measure of organizational performance [3].

Many companies consider quality as the core value of their organization and a critical factor for success in the competitive bids [4]. The research results have demonstrated that efforts to improve quality lead to an increase in product or service costs so that the quality improvement has its own costs. As a result, it is very important to conduct COQ analysis as an input to financial evaluation of quality improvement programs [5].

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http://dx.doi.org/10.28991/cej-2019-03091318

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The COQ is greatly important because of its extensive scopes [6]. Lam et al. (1994) claimed that the COQ could account for 8% to 15% of the total cost of the construction projects [4]. However, the literature review indicated that COQ, as an important quality management tool, is not applied in most quality management programs [7, 8].

According to research by Josephson et al. on construction projects, errors and problems computed 4.4% of total construction costs. In addition, these errors are equal to 7.1% of the working time or 34 minutes of a working day per person [4]. In 1978, these costs were estimated by the UK government at around 10% of the gross national product [9]. In the United States, direct costs due to reworks were estimated over 12% of the total cost of construction [9]. In Australia, it was concluded that every 1% more investment in prevention activities can reduce the total construction costs of failure from 2% to 10% [10].

The COQ measurement is not necessarily effective in improving quality. The COQ helps managers to evaluate their investment outcomes and will prepare their quality strategies and projects. The appropriate measuring instrument should be identified to measure the COQ. One of the most commonly used models for categorizing, detecting and measuring the COQ is the PAF approach, presented by Crosby in 1979 who divided the COQ into prevention, appraisal and failure costs. His work shows the relationship between failure costs and prevention and appraisal costs [4]. The RAF approach classifies the components as follows [4]:

- **The prevention costs**: the sum of all costs to avoid deficiencies before implementation including identifying the cause of the defect, undertaking corrective action to eliminate defects, personnel training, product or system redesign, the provision of new or altered equipment [4].
- **The appraisal costs**: the costs of monitoring, testing, or other costs incurred to ensure quality requirements and conformance of the product or process [4].
- **The internal failure costs**: the costs incurred due to product defects within the company and costs associated with defects found before delivering the product or service to the customer, including reworks, wastes, and repairs [4].
- **The external failure costs**: the costs incurred due to product defects after presenting by the company, including replacement of the product during the warranty period, loss of reputation of the company, handling of complaints and product repairs [4].

Although in the ASQ (American Society for Quality) references as a general guideline to all industries, most of the elements affecting the COQ are listed according to the above-mentioned classification system, these elements are presented in general, regardless of the type of project or construction system. In addition, the effect of each element on the final product quality is not determined separately [11].

Therefore, there is always a need to provide an appropriate platform for specifying, categorizing, and ranking the main COQ elements in mass-housing projects. Moreover, the determination of each parameter effect on final quality can help project managers to prioritize the parameters and elements affecting the COQ their associated costs.

In this study, the main parameters contributing in the COQ were listed on the basis of previous studies and semi-structured interviews with experienced experts and researchers in construction industry of Iran. Then, the impact of each parameter on final quality is determined and ranked in a comparative manner based on the Likert scale system from 1 to 5 [12].

The results of this research could be used as a basis for determining the quality costing system in the mass-housing projects. The ranking of COQ elements based on the impact on final quality can help decision-makers to focus on the most effective activities and it can result in increasing final quality and decreasing the quality costs.

2. Research Background

2.1. The COQ Studies and Implementation Analysis

As previously mentioned, the COQ is a tool for evaluating and measuring the efficiency of an organization or a process. Some organizations use COQ as a tool for weakness recognition and performance improvement consequently [13]. Moreover, some researchers exploit the COQ to evaluate a particular process or the performance of a system [14].

Al-Tmeemy and Rahman (2012) conducted a statistically qualitative survey to compare the benefits of implementing COQ and the requirements for conducting it between involved the parties. They divided the barriers into three cultural, systemic, and corporate classes, and stated that “management attention and increased quality awareness” are the highest advantages of measuring the quality costs [15].

Kiani and Shirovi Nezhad (2009) used a dynamic system for modeling the COQ [16]. Applied empirical studies were used to initialize their model. The study evaluated the impact of the costly factors on quality and came up with the following conclusions:
In general, the prevention activities have a greater impact on the COQ reduction compared to the appraisal activity. The prevention and appraisal activities are more effective in reducing the total COQ than when these activities work individually.

Sower and Quarels (2007) studied the role of COQ and quality growth in organization implementation. In their research, more than 30% of companies examined the COQ, in line with the previous research findings. They concluded that “the total quality of COQ will be reduced as processes of quality improvement, but the decreasing trend will be reduced” [17].

Omar et al. (2009), as well as Tye and Abdul Halim (2011), conducted studies on the implementation of COQ in the Malaysian construction industry. They evaluated the COQ levels and effects on the quality of achievements in the relevant industry unit. Their findings showed a high proportion of COQ to reduce the cost of non-functional and organizational level development [18].

2.2. The COQ Studies in Construction Industry

Although the COQ study has a history of more than 60 years, it is far newer in the field of construction industry, due to two main reasons:

1- Projects and, in particular, construction projects are unique, and it is difficult to establish a steady trend in these projects.

2- The time-consuming nature of the construction projects generally leads to costly and extensive research as a case study in this regard.

In the following section, the main studies have been reviewed in the field of the construction industry, and in particular mass-housing, focusing on newer studies.

Johnson (1995) probably conducted the first series of studies on the COQ in the construction industry. This study examined the methods for calculating the COQ in the construction industry implemented by a well-known government contractor in US, aiming to identify existing measures for costs of non-conformance functions in engineering operations and to suggest the best solutions applicable for use in the engineering employer unit. The information was gathered using the literature and telephone interviews with quality practitioners from major US corporations. Finally, different methods for measuring the costs of conformance (COC) and non-conformance (CONC) were evaluated. In addition to suggesting optimal methods, the role of the accounting unit, methods for collecting COQ data, subset reporting mechanism, and the findings of the interviews were also discussed [4].

Love and Irani (2003) developed prototype project management quality cost system (PROMQACS) in the construction projects. The results investigated and suggested the structure and information required for COQ classification system. To identify the information and management tools required to develop the PROMQACS system, the suggested system was tested and applied in two construction projects using a computer program. The proposed system was also used to determine the costs and the causes of the rework generally occurring in the projects. This research recommended that the project participants and particularly construction contractors can utilize PROMQACS to detect short comes in their project-related activities and consequently make the best decision to improve their project management system in the future. The advantages and limitations of this system were also identified in these studies [19].

Kazaz et al. (2005) investigated a mass-housing project in Elazığ, Turkey. The project included 3100 housing units, the construction of which lasted over 4 years. Their study revealed that the total COQ was averagely 32.36% of the total project cost. They also considered this value to be very high, which could be attributed to the poor executive project and the lack of internationally certified contractors [20].

Newton and Christian (2006) evaluated the COQ of construction projects and the impact of quality on construction costs. To this end, data related to the design costs, construction costs, operation costs, maintenance costs of 215 buildings were collected from all available databases from the Canadian Department of National Defense (DND). A measurement scale was developed to measure quality at all stages of initial building design, construction, operation and maintenance throughout the life cycle of the project. Analysis of variance (ANOVA) of the total annual costs in the first 20 years of the life cycle of buildings clearly indicated that the quality would have the greatest effect on total costs when the impact of other potential parameters, particularly the life cycle, was minimized or eliminated. It was also concluded that the quality and especially design quality would have the most significant impact on the maintenance costs [21].

Abdelsalam and Ghad (2009) investigated a mass-housing project in Dubai, UAE. The project included the construction of 291 multi-storied residences. The results revealed that the total COQ represents an average of 1.3% of the total costs of the projects. However, they could not calculate the external failure costs because their project was not yet handed over to the client. They also stated that the percentage of 1.3 is very low, and the reason could be that the
employer monitored the execution of the work on a daily basis using project management tool (PMT) and a consultant. This not only elevated the accuracy of contractors but also reduced their appraisal and prevention costs [13].

Love and Jafari (2013) assessed the effectiveness of a quality program during the initial 18 months of a monorail project in Iran. In this project, the quality cost system was operationally implemented. Ultimately, the failure cost was calculated to be 5% of the project’s contract value. Implementation of quality management program reduced this value down to about 2.78%, and 2.32% of the project’s contract value was attributable to appraisal costs. The major factors in reducing COQ in the failure subscale were the use of full-time quality management teams and repetitiveness of the activities. The active performance management and appraisal team and contractor monitoring have led to detect the errors and problems of the initial design before implementation. This raised the efficiency of the quality management system and improved the cost-cutting mechanism. In this project, the experiences of the contractor in the field of operations and analysis of COQ have proven to be promising in providing learning opportunities for other companies and consequently implementing quality improvement programs [22].

Jafari and Heravi (2014) investigated quality-related activities using 77 structured interviews in 60 mass-housing construction projects in Iran. In this study, the most important quality-related activities and COQ components were first identified. A model was developed to evaluate the total COQ of the studied projects by fitting the third-ordered curve to the extracted data. Then, cost-cutting potentials as the result of quality management obtained by COQ optimization were estimated based on the developed model. In fact, this research has taken a major step in comprehensive quality management (CQM) by developing the appraisal COQ model and suggesting an optimal COQ in mass-housing projects. Moreover, this model provided an optimal level of COQ and could yield significant cost-cutting in COQ and thus the total costs of the project [1].

Robfeld et al. (2015) evaluated the effectiveness and efficiency of quality management system indicators in some reputable German companies. They concluded that the high cost of data collection, problems of cost and benefit isolation, the lack of benefit expectancy, the lack of knowledge of methods, and the hard process of quantification of the quality-related benefits are the main problems in COQ implementation systems [23].

Alglawe et al. (2017) used the system dynamic approach to examine the effects of incorporating the opportunity cost into quality costing calculations in order to build a general framework within the supply chain. They concluded that when the opportunity cost is considered in the COQ model, the number of new customers and production units in supply chain decreases, which highlights the importance of the opportunity cost analysis in making decisions for the quality management strategies [24].

Glogovac and Flipovic (2018) expanded the level of knowledge about quality costing in active companies including both manufacturing and service-based companies. Their results confirm that companies which attribute to the fulfillment of certain requirements of ISO 9001:2015 for the adequacy of COQ management achieve better results [25].

2.3. Failure Costs Evaluation of the Construction Projects

The main researches carried out in the field of failure costs and especially rework costs in construction industry have been summarized in Table 1. As it is clear, the failure costs reported the table have not been estimated using the same method and each work has used its own method.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Country</th>
<th>Studied projects</th>
<th>Sample size</th>
<th>Percentage</th>
<th>Failure cost</th>
<th>Data resources</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burati et al. (1992)</td>
<td>US</td>
<td>Industrial</td>
<td>9</td>
<td>12.4</td>
<td>Direct costs</td>
<td>field data collection +</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>document inspection</td>
</tr>
<tr>
<td>Abdul-Rahman et al. (1996)</td>
<td>UK</td>
<td>Industrial</td>
<td>1</td>
<td>6</td>
<td>Cost of resources + time-</td>
<td>Inspection of related documents</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>related costs</td>
<td></td>
</tr>
<tr>
<td>Josephson and Hammarlund (1999)</td>
<td>Sweden</td>
<td>Building</td>
<td>7</td>
<td>3.2-4.9</td>
<td>Direct costs</td>
<td>Field data collection</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hall and Tomkins (2001)</td>
<td>UK</td>
<td>Building</td>
<td>1</td>
<td>5.8</td>
<td>Direct costs + Delay costs</td>
<td>field data collection +</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>document inspection</td>
</tr>
<tr>
<td>Kazaz et al. (2005)</td>
<td>Turkey</td>
<td>Building</td>
<td>3100</td>
<td>11.6</td>
<td>Internal and external failure costs</td>
<td>Inspection of related documents</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abdelsalam and Gad (2009)</td>
<td>UAE</td>
<td>Building</td>
<td>291</td>
<td>0.7</td>
<td>Internal failure costs</td>
<td>Inspection of related documents</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Love et al. (2010)</td>
<td>Australia</td>
<td>Infrastructure</td>
<td>115</td>
<td>10.3</td>
<td>Direct costs + Indirect costs</td>
<td>field data collection +</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>document inspection</td>
</tr>
<tr>
<td>Oyewobi et al. (2011)</td>
<td>Nigeria</td>
<td>Building</td>
<td>25</td>
<td>3.47</td>
<td>Direct costs</td>
<td>Inspection of related documents</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jafari and Love (2013)</td>
<td>Iran</td>
<td>Monorail</td>
<td>1</td>
<td>0.05</td>
<td>On-site costs</td>
<td>Field data collection</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jafari and Heravi (2014)</td>
<td>Iran</td>
<td>Monorail</td>
<td>1</td>
<td>0.05</td>
<td>Direct Costs</td>
<td>Inspection of related documents</td>
</tr>
</tbody>
</table>

Table 1. Percentage of Failure Cost in Proportion to Total Project Cost in Previous Studies
3. Research Methodology

This research involved different stages and different methods at each stage. The main parameters affecting the COQ have been specified from literature review and previous studies. Then, using semi-structured interview, the main parameters having the most impact on COQ of mass-housing projects have been selected from the list. After that, the parameters ranked and prioritized based on their relative importance index. These four main steps have been shown in Figure 1.

3.1. Semi-structured Interviews

To collect data associated with the parameters affecting the COQ in mass-housing projects, a thorough literature review was first carried out to identify a preliminary list of those involved. The studies reviewed all PAF elements in construction industries [1, 11, 19]. This resulted in the identification of an initial list of 64 parameters effective on the COQ. Semi-structured interviews were then conducted with 20 experts, consisting of three project manager, two deputy manager, four on-site discipline managers, four quality managers, and seven site engineer and experts. The reason for the combination of experts from different backgrounds was to provide a balanced view of the research topic. All these experts have sufficient working experience in the mass-housing industry and frequently deal with quality issues.

They were requested to identify COQ elements according to their own experience in the mass-housing construction project. In these interviews, the effective elements on the COQ identified from previous studies were listed and then some of them were deleted by questions and answers. In addition to these elements, the other parameters offered by the interviewee were added to them or merged with another row. Finally, the final list was used along with the original list in the next interview. This list, after completing and achieving the final parameters, was sent to the interviewees once again for comment in order to obtain their opinion. Therefore, the final list was approved by all interviewees.

Two conditions considered in selecting parameters: 1) having the greatest impact on the quality and 2) having the ability to measure, analyze, and review. For example, since it is impossible to calculate the cost of discredit caused by poor quality, despite the mention of this case and acknowledgment of its high impact, it was removed from the list.

Finally, this resulted in the identification of 17 COQ main elements, as summarized in Figure 2. The description of each parameter has been specified in Table 2.

Table 2. COQ elements description

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Description</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Project Quality management plan</td>
<td>Determination of requirements, expected quality level, tools and technics needed to reach this quality level, which will be prepared according to the nature and dimensions of the project at the beginning of the initial phase and before the start of the construction.</td>
<td>PR1</td>
</tr>
<tr>
<td>2</td>
<td>Work instructions, method statements and workflow design</td>
<td>Design and provision of the method statements, the operation flowchart, the work process and the order of the operation, as well as the quality control check points.</td>
<td>PR2</td>
</tr>
<tr>
<td>3</td>
<td>Quality management system</td>
<td>Determination, designing and producing of checklists, tolerances, inspection and test programs and and other tools for product quality control process.</td>
<td>PR3</td>
</tr>
<tr>
<td>4</td>
<td>Use of high-quality human forces</td>
<td>Finding and using experienced and high-quality experts for the construction process on the site.</td>
<td>PR4</td>
</tr>
<tr>
<td>5</td>
<td>In-service training of the project team</td>
<td>Includes all training for the purpose of improvement in the quality of personnel work and thus achieve a higher quality product.</td>
<td>PR5</td>
</tr>
<tr>
<td>No.</td>
<td>Description</td>
<td>Description</td>
<td>PR/EF</td>
</tr>
<tr>
<td>-----</td>
<td>-----------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>6</td>
<td>Searching, evaluation and selection of competent suppliers and subcontractors</td>
<td>Evaluation, classifying and grading of the suppliers and contractors before using in the project. This will allow the use of quality materials as well as qualified contractors for quality work.</td>
<td>PR6</td>
</tr>
<tr>
<td>7</td>
<td>Quality control team activity in prevention</td>
<td>Preventive actions and other related works done by the quality control team to improve the product quality before construction</td>
<td>PR7</td>
</tr>
<tr>
<td>8</td>
<td>Design and Implementation of motivation system (reward and penalty)</td>
<td>Designing, launching, monitoring and controlling of an effective motivation system to conduct staff doing their works with maximum possible quality. This system will contain all materials (cash and both of reward and penalties).</td>
<td>PR8</td>
</tr>
<tr>
<td>9</td>
<td>Laboratory</td>
<td>Including all activities in planning, preparation, launching, producing test data, tests running and recording results, reporting, results evaluating etc. and generally all activities related to test of works done.</td>
<td>AP1</td>
</tr>
<tr>
<td>10</td>
<td>Quality control forces</td>
<td>It contains all quality control operations done with Quality staff.</td>
<td>AP2</td>
</tr>
<tr>
<td>11</td>
<td>Inspection and getting approve of External organizations</td>
<td>All inspections and quality control processes done with external organizations which have to approve the designs (like Firefighting organization, Construction engineering organization etc.) belong to this group.</td>
<td>AP3</td>
</tr>
<tr>
<td>12</td>
<td>Design correction</td>
<td>This section relates to services provided by the engineering department for designs correction due to computational or execution problems.</td>
<td>IF1</td>
</tr>
<tr>
<td>13</td>
<td>Non-Conformances and related reworks</td>
<td>All works done due to low quality of products which cannot passed quality limits. It contains all destroying works done, corrective actions and reworks.</td>
<td>IF2</td>
</tr>
<tr>
<td>14</td>
<td>Re-evaluation and re-tests due to non-conformances</td>
<td>All appraisal works happen in the reworks which have not passed quality limitations first time.</td>
<td>IF3</td>
</tr>
<tr>
<td>15</td>
<td>Legal proceeding, complaints, and handling claims</td>
<td>All activities related to handling claims and complaints or courts that have been handed after the delivery of the product.</td>
<td>EF1</td>
</tr>
<tr>
<td>16</td>
<td>Penalties for poor quality</td>
<td>Includes all penalties imposed by the employer due to poor quality of service.</td>
<td>EF2</td>
</tr>
<tr>
<td>17</td>
<td>Corrections and repairs in warranty period</td>
<td>All costs and activities incurred during the guarantee period by the contractor. Both of continuous costs (on-site staff during the guarantee period) or case costs (due to probable corrective actions.</td>
<td>EF3</td>
</tr>
</tbody>
</table>

![Figure 2. The main COQ elements specified in the mass-housing projects](image-url)
3.2. Questionnaire Design

The questionnaire is divided into two main parts. The first part is related to general information about responder persons like their personal information, experience in mass-housing projects and their opinions about the COQ elements of mass-housing projects they have experienced. The second part includes the list of the identified COQ elements. The respondents were asked to score the elements based on the impact on the final quality cost of the related construction projects. The following five levels of scoring was adopted using Likert scale ‘Very High Impact’ (5 points), ‘High impact’ (4 points), ‘Moderate’ (3 points), ‘Low impact’ (2 points) and ‘very Low or no impact’ (1 point) on final quality of the products.

A total of 200 questionnaires were distributed by e-mail, social networks, and project sites. Although the respondents asked to use the online questionnaire which was designed and launched in Google infrastructure, some experts preferred to use paper or word version for it. So the questionnaires gathered by e-mail or paper have filled in the related website by us. Over a period of 1 month, 148 questionnaires were returned, which comprised 72 online, 45 emails and 41 questionnaires collected from construction sites. Of these, 28 were discarded because of incomplete or invalid information provided by the respondents. The remaining 120 valid questionnaires are used for analysis, representing a very good response rate of 60% (Table – shows the details of related data), which is enough for a reliable analysis [26].

4. Results and Discussions

4.1. Method of Data Analysis

The Relative Importance Index (RII) method has been used to logically evaluate and rank the COQ elements according to their degree of importance. The impact on final quality will be measured using the formula presented in Equation 1 [27]:

$$RII = \frac{\sum W}{A \times N}$$

(1)

Where RII is the Quality index, W: weight of each element (determined by questionnaire and variable from 1: very low impact up to 5: very high impact), A: highest possible weight (here is 5), and N: the total number of respondents.

4.2. Analysis and Ranking of COQ Elements

The survey’s results have been analyzed with the Statistical Package for the Social Sciences (SPSS Version 17.0) Statistics [28]. Cronbach’s test is used to measure the internal reliability of the questionnaire. The values of alpha for prevention, appraisal, and internal and external failure-related activities groups are 0.714, 0.702, 0.729, and 0.763, respectively, which were all higher than the acceptable threshold of 0.7 [27].

The RII calculated for all elements in each PAF groups from the questionnaires which are described below.

4.3. Prevention Elements Ranking

Table 3 shows the RII and ranking of each COQ elements in the prevention activities group. 8 elements were considered in this group. The table shows that the top three elements which have more impact on final product quality in mass-housing projects are: quality management system (RII = 90.9%), using of high-quality human resources (RII = 89.8%) and selection of competent suppliers and subcontractors (RII = 88.0%). As expected, the prevention activities have the most impact on quality in construction projects. The items related to this group ranged from 73.9% to 90.9% (severity level ranges from high to very high).

The results indicate that using high-quality staff could be more effective than the training of ordinary staff. The results also prove the fact that motivation systems cannot guarantee the low-quality product removing.

<table>
<thead>
<tr>
<th>Rank</th>
<th>RII</th>
<th>Standard Deviation</th>
<th>COQ Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.90</td>
<td>0.67</td>
<td>Quality Management System</td>
</tr>
<tr>
<td>2</td>
<td>0.89</td>
<td>0.71</td>
<td>Using high-quality human resources</td>
</tr>
<tr>
<td>3</td>
<td>0.88</td>
<td>0.67</td>
<td>Searching, evaluation and selection of competent suppliers and subcontractors</td>
</tr>
<tr>
<td>4</td>
<td>0.85</td>
<td>0.81</td>
<td>Work Instruction, method statement and workflow design</td>
</tr>
<tr>
<td>5</td>
<td>0.85</td>
<td>0.66</td>
<td>Project Quality Management plan</td>
</tr>
<tr>
<td>6</td>
<td>0.80</td>
<td>0.62</td>
<td>Quality control team activity in prevention</td>
</tr>
<tr>
<td>7</td>
<td>0.79</td>
<td>0.82</td>
<td>In-service training of the project team</td>
</tr>
<tr>
<td>8</td>
<td>0.75</td>
<td>0.97</td>
<td>Design and implementation of motivation systems</td>
</tr>
</tbody>
</table>

0.84
4.4. Appraisal Elements Ranking

Table 4 shows the RII and ranking of each COQ elements in the appraisal activities group. 3 elements were considered in this group. The table shows that laboratory activities (RII = 82%), have more impact on quality costs compared to quality control processes (RII=74%). The reason can be related to the reality that the most impacts of quality control process on quality costs have been considered in prevention activities in the item of “Quality Control team activities in prevention”. As it is clear, the “Assessment of external organizations” has not most impact on final quality.

Table 4. Ranking of COQ elements of mass housing projects in Appraisal group

<table>
<thead>
<tr>
<th>Rank</th>
<th>RII</th>
<th>Standard Deviation</th>
<th>COQ Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.82</td>
<td>0.64</td>
<td>Laboratories</td>
</tr>
<tr>
<td>2</td>
<td>0.74</td>
<td>0.68</td>
<td>Quality control processes</td>
</tr>
<tr>
<td>3</td>
<td>0.66</td>
<td>0.94</td>
<td>Assessment of external organizations</td>
</tr>
</tbody>
</table>

4.5. Failure Elements Ranking

Table 5 shows the RII and ranking of each COQ elements in the failure activities group. Each failure group (internal and external) has 3 elements and totally 6 elements were considered in this group. Although as it was expected, this group has the least impact on COQ, some activities in the internal failure group may play a very important role in final product quality. The table shows that “Redesigns and design corrections” as well as “Reworks”, are the most important items in the failure group (RII=79%). The items related to this group ranged widely from 58% to 79% (severity level ranges from low to high).

From the table, it is quite clear that although “Legal proceeding, complaints, and claim handling” could be very costly in the projects, their impacts on quality evaluated lower than other elements.

Table 5. Ranking of COQ elements of mass housing projects in the Failure group

<table>
<thead>
<tr>
<th>Rank</th>
<th>RII</th>
<th>Standard Deviation</th>
<th>COQ Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.79</td>
<td>0.84</td>
<td>Redesigns and design corrections</td>
</tr>
<tr>
<td>2</td>
<td>0.79</td>
<td>0.79</td>
<td>Reworks</td>
</tr>
<tr>
<td>3</td>
<td>0.72</td>
<td>0.77</td>
<td>Reassessments and inspection due to rework</td>
</tr>
<tr>
<td>4</td>
<td>0.69</td>
<td>0.91</td>
<td>Corrections and repairs in warranty period</td>
</tr>
<tr>
<td>5</td>
<td>0.66</td>
<td>1.24</td>
<td>Penalties for poor quality</td>
</tr>
<tr>
<td>6</td>
<td>0.58</td>
<td>0.97</td>
<td>Legal proceeding, complaints, and claim handlings</td>
</tr>
</tbody>
</table>

4.6. Overall COQ Elements Ranking

The relative importance index and ranking of all investigated 17 COQ elements in Mass-housing construction projects are listed in Table 6. As it was expected, the prevention activity group elements have the most impact on quality compared to other groups.

Table 6. Overall COQ elements ranking in mass housing projects

<table>
<thead>
<tr>
<th>Rank</th>
<th>Group</th>
<th>Standard Deviation</th>
<th>COQ Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PR</td>
<td>0.66</td>
<td>Project Quality Management plan</td>
</tr>
<tr>
<td>2</td>
<td>PR</td>
<td>0.67</td>
<td>Quality Management System</td>
</tr>
<tr>
<td>3</td>
<td>PR</td>
<td>0.71</td>
<td>Using high-quality human resources</td>
</tr>
<tr>
<td>4</td>
<td>PR</td>
<td>0.67</td>
<td>Searching, evaluation and selection of competent suppliers and subcontractors</td>
</tr>
<tr>
<td>5</td>
<td>PR</td>
<td>0.81</td>
<td>Work Instruction, method statement and workflow design</td>
</tr>
<tr>
<td>6</td>
<td>AP</td>
<td>0.64</td>
<td>Laboratories</td>
</tr>
<tr>
<td>7</td>
<td>PR</td>
<td>0.62</td>
<td>Quality control team activity in prevention</td>
</tr>
<tr>
<td>8</td>
<td>IF</td>
<td>0.84</td>
<td>Redesigns and design corrections</td>
</tr>
<tr>
<td>9</td>
<td>IF</td>
<td>0.79</td>
<td>Reworks</td>
</tr>
<tr>
<td>10</td>
<td>PR</td>
<td>0.82</td>
<td>In-service training of the project team</td>
</tr>
<tr>
<td>11</td>
<td>PR</td>
<td>0.97</td>
<td>Design and implementation of motivation systems</td>
</tr>
<tr>
<td>12</td>
<td>AP</td>
<td>0.68</td>
<td>Quality control processes</td>
</tr>
</tbody>
</table>
The following results can be derived from the values shown in Table 6:

- As expected, the prevention and external failure activities had the highest and the least effects on the quality, respectively. However, the impact of internal failure costs on the final product quality has been particularly high.
- According to the interviewees, the quality management system from the prevention activities has identified as the most effective element, and trials, complaints and claims handling from the external failure section as the least influential element in the final product quality.
- As a result of interviews, the use of specialist and high-quality human forces is far more effective than in-service training of regular and inexperienced forces.
- Based on the results of the interviews, the design and implementation of the incentive system will not have much effect on the final product quality.
- The activities related to quality control operations by the quality control team will have a significant impact on the final product quality.
- According to the results obtained, the evaluation of external organs has no particular effect on the final product quality, and the internal sensitivity of the company has far more effect on providing high-quality of the product compared to the appraisal of external organizations.

### 4.7. Group Ranking

The ranking of the main groups of COQ in mass-housing construction projects and comparison of the results with Jafari and Heravi (2014) is shown in Table 7. Unexpectedly the internal failure group ranked better than appraisal group. It happened due to the low impact of “Assessment of external organizations” which has decreased the average RII for the appraisal group. This factor has not been considered in Jafari and Heravi (2014). So the rank of appraisal group in that work has been evaluated more than the present study. In addition some effective factors in prevention group like “Using high-quality human resources” which has evaluated as a high impactful factor on final product quality has not been considered in Jafari and Heravi (2014). On the other hand, the activities of quality control staff has been divided into two main group of “Quality control team activity in prevention” which has been considered in prevention group, and “Quality control processes” which has been considered in prevention group. Hence, the weight of prevention activities has been raised in the present study compared to the mentioned work.

Although some internal failure activity programs like “Redesigns” or “Reworks” could be very costly compared to prevention and appraisal activities, they can remove most of the low quality products and have a high impact on the final quality consequently.

<table>
<thead>
<tr>
<th>COQ Group</th>
<th>Jarari and Heravi (2014)</th>
<th>Present Study</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RII</td>
<td>Rank</td>
</tr>
<tr>
<td>Prevention</td>
<td>0.73</td>
<td>2</td>
</tr>
<tr>
<td>Appraisal</td>
<td>0.81</td>
<td>1</td>
</tr>
<tr>
<td>Internal failure</td>
<td>0.72</td>
<td>3</td>
</tr>
<tr>
<td>External failure</td>
<td>0.51</td>
<td>4</td>
</tr>
</tbody>
</table>
5. Conclusion

In this article, the main elements of COQ were detected and extracted in mass housing projects. Subsequently, these elements were prioritized by experts in the industry using questionnaires and semi-structured interviews. Based on PAF method, the COQ elements classified under 4 groups: prevention, appraisal, internal failure, and external failure.

The results indicate that the prevention group has the most impact and external failure group elements have the least impact on the final quality of the products.

According to the results, the use of specialist and high-quality human forces is far more effective than in-service training of inexperienced forces. Corrective actions of non-conformities and design improvements have a significant impact on final product quality. Accordingly, the re-evaluation of corrective actions is far less important than the primary assessment of the activities undertaken.

The ranking of COQ elements based on the impact on final quality can help project managers to select the most effective activities in the limited budget conditions. This can help project managers to maximize the quality and minimize costs simultaneously.

6. Acknowledgements

Authors would like to thank and appreciate all the people who participated and collaborated in interviewing and sending questionnaires, especially directors and experts at the Kayson Company, as one of the leading companies in construction projects.

7. Conflict of Interest

The authors declare no conflict of interest.

8. References


Effect of Soaking and Non-soaking Condition on Shear Strength Parameters of Sandy Soil Treated with Additives

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Received 10 January 2019; Accepted 30 April 2019

Abstract

The present paper aims to improve shear strength parameters: cohesion (c), and angle of internal friction (\(\phi\)) for sandy soil treated by additives before and after soaking. The samples of sandy soil were obtained from Karbala city and then classified as poorly graded sand (SP) with relative density \(D_r\) (30\%) according to the system of (USCS). The experiment has three stages. In the first stage, the soil was treated with three different percentages of cement (3, 5 and 7\%) of dry weight for the soil with three different percentages of water content (2, 4 and 8\%) in each above percentage of cement, while the second stage includes (2\%) of lime from soil weight mixed with each different percentage of cement. In the third stage, (50\%) of polymer of cement weight was mixed with each different percentage of cement. An analysis of behavior sandy soils treated by additives was carried out with the Direct Shear Tests. All the samples were cured (3) days before and after soaking. The results of the experiment showed that increase in shear strength parameters of sandy soil; especially the angle of internal friction with the rate value (16.6\%) of cement only, (21.88\%) of cement with lime, (20.3\%) of cement with the polymer before soaked condition. After soaking condition, it was increased with the rate value (14.3\%) with cement only, (23.57\%) of cement with lime, and (15.38\%) of cement with the polymer as compared with soil in the natural state.

Keywords: Additives Soil; Sandy Soils; Shear Strength Parameters; Cement; Lime; Polymer.

1. Introduction

Ground improvement is a process that aims to enhance the engineering properties of the soils and generate an improved constriction material by increasing soil strength, durability, stiffness, and decreasing permeability and compressibility of sandy soils. Additives materials are one of the most important methods to improve the engineering properties of soil that are used to improve the engineering performance. For example, Lime stabilization and cement stabilization are the two common additives of the material methods. These additives materials are categorized as traditional and non-traditional materials. The combination of traditional additives includes (cement, lime, fly ash, and bitumen materials, while non-traditional additives includes various combinations such as (enzymes, polymer, resins, acid, calcium chloride, sodium chloride, and fiber reinforcement). Many researchers relatively have studied the effect of the stabilization agent on shear strength parameters Mitchell [1] showed that stabilization agents increase the effect of cohesion. Other researchers such as Lo, SR, and S PR Wardani [2] found out that the soil treated by cement presented a significant increase in both internal friction angle and cohesion. While Balmer [3] concluded that the value of the internal friction angle varies from (36.1° to 43.8°) for fine and coarse-grained stabilized soils. While the shear strength parameters: cohesion(c), and angle of internal friction (\(\phi\)) increase with increasing cement content according to Al-Aghbari et al. [4] and Shooshpasha, and Reza [5]. Ziaie-Moayed et al. [6] conducted research to improve the saline soils before and after soaking by using cement and polymer on shear strength and they concluded ;(1) the strength of soil
specimens are increased by adding cement and resin epoxy and (2) saline materials have an effect on the strength of the soil specimens; in addition, the shear strength of the soil decreased after soaking. Das et al. [7] conducted research in order to improve shear strength parameters of sandy soils by the use of hair fiber as reinforcing the material. Their results showed that almost (11.5%) of enhancement in shear strength parameter is obtained when hair fiber reinforcement is used. Ahmed et al. [8] investigated the behavior of soil-cement and soil-lime mixture in order to improve the engineering properties of the soil. The study was focused on three dimensions: (1) the effect of their mixture, (2) the effect of the mixture percentages, and (3) the effect of curing time; the results show that stabilizer with a higher percentage of mixture gives a higher increase in shear strength and the strength increases with more curing time. Yousuf [9] investigated the effect of cement grouting with different percentages of the water-cement ratio (W:C) and filler materials to improve shear strength parameters of sandy soil. He founded that when the water-cement ratio decrease, the shear strength parameters increases. A recent study was conducted by Pakbaz MS [10] in studying the effect of microbial –included calcite precipitation (MICP) treatment on the shear strength parameters of sandy soil. This researcher stated that the increase in the cohesion intercept was more significant than the increase in the angle of internal friction. While Avci, and Mollamahmutoğlu [11] investigated the effect of synthesis of sodium silicate manide and sodium silicate –glyoxal grouted sand on shear strength parameters under wet cured and air dried. The results showed that the shear strength parameters of wet cured sand decreased with time more than dried sand samples.

2. Methodology

2.1. Physical Properties of Sandy Soil

The samples of sandy soil were obtained from Karbala city. The physical properties of the sand included specific gravity, grain size distribution analysis as in the Figure 1, and then classified as poorly graded sand (SP) according to (USCS). Maximum dry unit weight ($\gamma_{\text{max}}$) and minimum dry unit weight ($\gamma_{\text{min}}$). The results are shown in the Table 1.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Standard of the test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity (Gs)</td>
<td>2.68</td>
<td>ASTM D854</td>
</tr>
<tr>
<td>D10, mm</td>
<td>0.127</td>
<td>ASTM D422 and D2487</td>
</tr>
<tr>
<td>D30, mm</td>
<td>0.210</td>
<td></td>
</tr>
<tr>
<td>D60, mm</td>
<td>0.339</td>
<td></td>
</tr>
<tr>
<td>Coefficient of uniformity (Cu)</td>
<td>2.67</td>
<td></td>
</tr>
<tr>
<td>Coefficient of curvature (Cc)</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td>Type of soil</td>
<td>SP</td>
<td></td>
</tr>
<tr>
<td>Maximum dry unit weight (kN/m³)</td>
<td>17.48</td>
<td>ASTM D4253</td>
</tr>
<tr>
<td>Minimum dry unit weight (kN/m³)</td>
<td>14.91</td>
<td>ASTM D4254</td>
</tr>
<tr>
<td>Maximum void ratio($e_{\text{max}}$)</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>Minimum void ratio($e_{\text{min}}$)</td>
<td>0.53</td>
<td></td>
</tr>
<tr>
<td>Relative density (%)</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Angle of internal friction (φ°)</td>
<td>32°</td>
<td>ASTM D3080</td>
</tr>
<tr>
<td>Cohesion (c)</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

Figure 1. Grain size distribution curve of the sand
2.2. Chemical and Physical Properties of Additives Materials

**Cement:** The type of cement used in the research is sulfate resistant cement (Type II), this cement is produced by Tasluja cement factory. The chemical and physical properties of cement are listed in Table 2.

<table>
<thead>
<tr>
<th>Table 2. Chemical and Physical Properties of the Cement</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Property</strong></td>
</tr>
<tr>
<td>Compressive strength after 3 days (Mpa)</td>
</tr>
<tr>
<td>Compressive strength after 7 days (Mpa)</td>
</tr>
<tr>
<td>Time of initial setting (hour)</td>
</tr>
<tr>
<td>Time of final setting (hour)</td>
</tr>
<tr>
<td>SiO$_2$ (%)</td>
</tr>
<tr>
<td>CaO (%)</td>
</tr>
<tr>
<td>Al$_2$O$_3$ (%)</td>
</tr>
<tr>
<td>Fe$_2$O$_3$ (%)</td>
</tr>
<tr>
<td>MgO (%)</td>
</tr>
<tr>
<td>SO$_3$ (%)</td>
</tr>
<tr>
<td>C$_3$A (%)</td>
</tr>
<tr>
<td>LOI (%)</td>
</tr>
</tbody>
</table>

**Lime:** Hydrated lime was used in the experiment. The following Table 3 shows the chemical and physical properties of lime that it supplied by the manufactory.

<table>
<thead>
<tr>
<th>Table 3. Chemical and Physical Properties of Lime</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Properties</strong></td>
</tr>
<tr>
<td>Chemical formal</td>
</tr>
<tr>
<td>Appearance</td>
</tr>
<tr>
<td>L.O.I</td>
</tr>
<tr>
<td>SiO$_2$ %</td>
</tr>
<tr>
<td>Al$_2$O$_3$ %</td>
</tr>
<tr>
<td>CaO %</td>
</tr>
<tr>
<td>MgO %</td>
</tr>
<tr>
<td>SO$_3$ %</td>
</tr>
<tr>
<td>Density gm/cm$^3$</td>
</tr>
</tbody>
</table>

**Polymer:** The chemical and physical properties of polymer type (cebex 100) are listed in Table 4.

<table>
<thead>
<tr>
<th>Table 4. Chemical and Physical Properties of Polymer</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Properties</strong></td>
</tr>
<tr>
<td>Composition</td>
</tr>
<tr>
<td>Appearance</td>
</tr>
<tr>
<td>Odour</td>
</tr>
<tr>
<td>Relative density at 20 C($^\circ$g/cm$^3$) bulk</td>
</tr>
<tr>
<td>Water solubility</td>
</tr>
<tr>
<td>PH</td>
</tr>
</tbody>
</table>

2.3. Characteristics of the Direct Shear Test

The test of shear strength of the soil was carried out in the geotechnical laboratory/department of the Civil Engineering / University of Baghdad. The researcher used Standard Direct Shear which has a box with dimensions 6×6 cm. A vertical load was applied to the square specimen through a static weight hanger and the sample was sheared by applying the horizontal force that causes the two halves of the box to move relative to each other. The constant rate of strain 1.2
mm/min. The magnitude of proving ring with capacity 2 kN. The precision of dial gage for vertical and horizontal deformation was 0.002 and 0.01 mm respectively as shown in Figure 2. The test was performed with the normal stresses of 54.5, 109, and 218 kPa.

2.4. Sample Preparation

The study sample includes 26 of untreated and treated soil before and after soaking. In the beginning, the test was done on the untreated soil according to ASTM D3080 [17], and then the test was carried out with treated soil. The treated soil by additives contains; (3, 5 and 7%) of cement from dry weight of the soil in the first stage. 2% of lime from the dry weight of the soil mixed with each percentage of cement in the second stage. In the last stage, (50%) of polymer from weight cement mixed with each percentage of cement. During the test, three water content (W:C) (2, 4 and 8%) were used with each percentage of cement before soaking, and then the optimum water content with lime and polymer was used before and after soaking. Samples treated were covered with two layers (nylon and cellophane sheet) in order to maintain the moisture content during (3) days of the treatment period. In the case of the soaking condition, the samples were soaked for (1) day, and then the test was conducted, (see Figure 3).
3. Direct Shear Test Results and Discussion

3.1. First Group Results

The first group included dry sand before soaking test condition with a relative density of (30%). The results showed that the cohesion \( c \) is equal to zero, while the angle of internal friction \( \phi \) is equal to \( 32^\circ \). After soaking the soil for (1) day, the cohesion slightly increased to (2.5) kPa, while the angle of internal friction decreased to (28\(^\circ\)). This is due to the solubility of salts in the soil with water causes decrease in angle of internal friction. Figure 4 shows the relationship between normal stress versus maximum shear stress.

![Figure 4](image-url)  
Figure 4. Shows shear stress versus normal stress for untreated samples of soil before and after soaking

3.2. Second Group Results

This group is included improved soil by cement with three percentages (3, 5 and 7\%) of dry weight of soil before soaking condition, and then it was cured with three different percentages of water content (2, 4 and 8\%) for (3) days. The relationship between normal stresses versus maximum shear stress is shown in Figures 5, 6 and 7. From the experimental work, the results of effect (W:C) with soil treated by cement on the shear strength parameter explained that the angle of internal friction increased when soil was treated by cement with 4\% (W:C) of the rate value (between untreated soil and treated soil by cement) between (10-16.6\%); therefore the optimum moisture content (OMC) is 4\%. While the cohesion increased with 8\% of (W:C) with a value range between (6.3-8.5) kPa as compared with the untreated sample. The results are illustrated in the Table 5.

Shear strength parameters: \( c \) and \( \phi \) increase with increasing cement content. Thus, the soil treated by 7\% of cement with 4\% OMC is the optimum value as a result of increasing the shear strength parameters \( c \) and \( \phi \) to (7 kPa, 16.6\%) respectively, as they are shown in the Figures 8 and 9. This due to increase hydration of cement led to increase the bonding force between grains of soil cement. These findings are also introduced by [4-6], and [18-20]. Later, Laguros J.G [21] indicated that the cohesion as well as the angle of internal friction increase when the soils are treated with cement, the increase being higher for the granular soils than for the fine grained soils.

![Figure 5](image-url)  
Figure 5. Shows shear stress versus normal stress for soil sample treated by cement at 2\% W:C before soaking condition
Figure 6. Shows shear stress versus normal stress for soil sample treated by cement at 4% W:C before soaking condition.

Figure 7. Shows shear stress versus normal stress for soil sample treated by cement at 8% W:C before soaking condition.

Figure 8. Shows the relationship between the percentages of cement with the angle of internal friction before soaking condition.
Figure 9. Shows the relationship between the percentages of cement with cohesion before soaking condition.

Table 5. Shows the Results of Direct Shear Test on Sandy Soil treated by Cement with Different (W:C) Before Soaking Condition

<table>
<thead>
<tr>
<th>W:C with the percent of cement</th>
<th>The result of the direct shear test</th>
<th>% Increasing of $\phi$ treated soil by cement before soaked</th>
</tr>
</thead>
<tbody>
<tr>
<td>% W:C</td>
<td>% of Cement</td>
<td>c (kPa)</td>
</tr>
<tr>
<td>Untreated soil</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>2.5</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>35.2</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>5.4</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>3</td>
<td>6.3</td>
<td>34.4</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>7</td>
<td>8.5</td>
<td>35.8</td>
</tr>
</tbody>
</table>

3.3. Third Group Results

The third group of samples contained soil treated by cement with three percentages (3, 5 and 7%) of dry weight of soil, and treated with (OMC) after soaking condition. The relationship between normal stresses versus maximum shear stress is shown in Figure 10. The experimental work showed the effect of soil treated by cement on the shear strength parameters after soaking; the cohesion has value range between (9.5-12 kPa), and the rate of increase in cohesion between (280-380%). While the angle of internal friction is increased with the rate value range between (7.14-14.3%), as compared with untreated soil after soaking (see Table 6). In the soaking condition, when cement content increased, shear strength parameters increased as compared with untreated soil sample; therefore the soil treated by 7% of cement is the optimum content as it is shown in the Figures 11 and 12. This is due to an increase in the reaction when cement is mixed with water led to increase hydration of cement to fill particles of soil with water. Thus, the shear strength parameters decreased as compared with the samples before soaking condition, and this was also observed by [5].
Figure 10. Shows shear stress versus normal stress for soil sample treated by cement at 4% OMC after the soaking condition.

Figure 11. Shows the relationship between the percentages of cement with the angle of internal friction of soil treated at 4% OMC before and after the soaking condition.

Figure 12. Shows the relationship between the percentages of cement with the cohesion of soil treated at 4% OMC before and after the soaking condition.
Table 6. Shows the Results of Direct Shear Test on Sandy Soil Treated by Cement after Soaking Condition

<table>
<thead>
<tr>
<th>% Cement content with OMC</th>
<th>The result of the direct shear test</th>
<th>%Increasing $\phi$ of treated soil by cement under the soaked condition</th>
<th>%Increasing $c$ of treated soil by cement under the soaked condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated soil</td>
<td>2.5</td>
<td>28</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>9.5</td>
<td>30</td>
<td>7.14</td>
</tr>
<tr>
<td>5</td>
<td>10.6</td>
<td>31.5</td>
<td>12.5</td>
</tr>
<tr>
<td>7</td>
<td>12</td>
<td>32</td>
<td>14.3</td>
</tr>
</tbody>
</table>

3.4. Fourth Group Results

The fourth group included improved sandy soil of mixture 2% of lime added with each percentage of cement, and then treated with 4% OMC before soaking condition. The relationship between normal stresses versus shear stress is illustrated in (Figure 13). The experimental results of shear strength parameter showed that the cohesion has value range between (3.6-11.5 kPa) and the rate value of the increase in the angle of internal friction (between untreated soil and treated soil by mixing cement with lime) between (18.75-21.88%) as it is explained in the Table 7. Shooshpasha, and Reza [5] observed that the shear strength parameters: cohesion and internal friction angle increase with increasing lime Portland cement content. The substantial increase in cohesion is evident than internal friction angle.

From the results, the comparison between shear strength treated by cement only and shear strength parameters treated by mixture cement with lime before soaking can be summarized as follows:

1. Shear strength of soil treated by mixing cement with lime was increased more than the soil treated by cement only. The shear strength parameters were increased due to the pozzolanic activity of lime from the reaction of the calcium in the lime and cement with soil particles this led to increase bonding between the additives and the particles of soil, this was also noted by [8], and [9]. While the soil treated by 5% of cement with 2% lime is the optimum value because of increasing the rate value of shear strength parameters; the cohesion was (20.4%), and the angle of friction is (5.75%). Thus, it was more than soil treated by cement only.

2. The similar results, in the case of the soil treated by 7% of cement with 2% of lime, showed an increase in shear strength parameter more than soil treated by cement only. In case of soil treated by 3% of cement with 2% of lime also showed an increase in the rate value of the angle of internal friction to (7.25%), while it decreased in the cohesion to (10%). The effect of lime with cement to improve the shear strength parameters are shown in Figures 15 and 16.

![Figure 13. Shows shear stress versus normal stress for soil sample treated by mixing of cement with lime at 4% of OMC before soaking condition](image-url)
Table 7. The Results of Direct Shear Test for Soil Treated by Mixture Cement with Lime at 4% OMC Before soaking Condition

<table>
<thead>
<tr>
<th>Mixed type</th>
<th>Results of the direct shear test</th>
<th>% Increasing in $\phi$ for treated soil</th>
<th>Different in $%\phi$ for treated soil by cement and by cement with lime</th>
<th>% Rate of $c$ for treated soil by cement and by cement with lime</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated soil</td>
<td>0</td>
<td>32</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3% Cement + 2% Lime</td>
<td>3.6</td>
<td>38</td>
<td>18.75</td>
<td>7.95</td>
</tr>
<tr>
<td>5% Cement + 2% Lime</td>
<td>6.5</td>
<td>38.6</td>
<td>20.63</td>
<td>5.75</td>
</tr>
<tr>
<td>7% Cement + 2% Lime</td>
<td>11.5</td>
<td>39</td>
<td>21.88</td>
<td>4.56</td>
</tr>
</tbody>
</table>

3.5. Fifth Group Results

This group included improved sandy soil by mixture 2% of lime added with each percentage of cement, and then treated with 4% of OMC after soaking condition. The relationship between normal stresses versus shear stress was shown in Figure 14. The experimental work showed the results of shear strength parameters were: the cohesion has a value range between (2.5-15.6 kPa) and the increasing in the rate value between (380-524%). The angle of internal friction has the rate value of increasing (between untreated soil and treated soil by mixing cement with lime) between (17.86-23.57%), all results shown in the Table 8.

To make a comparison between shear strength treated by cement only and shear strength parameters treated by mixing (cement with lime) after soaking condition, the researcher noticed that:

- Shear strength of soil treated by mixing cement with lime increased more than the soil treated by cement only. The shear strength parameters increased as a result of the ability of lime to absorb the water and to complete the reaction of additives with particles of the soil by the presence of water. Soil treated by 7% cement with 2% lime is the optimum value as a result of increasing the rate value of shear strength parameters; the cohesion was (8.14%) and the angle of friction was (30%), more than soil treated by cement only.

- The shear strength parameters of the soil treated by (3 and 5%) of cement and 2% of lime were increased more than the soil treated by cement only. The effect of lime with cement to improve the shear strength parameters of the sandy soil increases when cement content increase (as it is shown in Figures 15 and 16).
Figure 15. Shows the relationship between the percentages of mixing cement with lime and the angle of internal friction for soil treated at 4% OMC before and after soaking conditions

Figure 16. Shows the relationship between the percent of mixing cement with lime and the cohesion for soil treated at 4% OMC before and after soaking conditions

Table 8. Illustrates the results of Direct Shear Test for Soil Treated by Mixture Cement with Lime at 4% of OMC after Soaking Condition

<table>
<thead>
<tr>
<th>Mixed type</th>
<th>Results of the direct shear test</th>
<th>% Increasing in $\phi$ for treated soil</th>
<th>% Increasing in $c$ for treated soil</th>
<th>% Different in $\phi$ for treated soil by cement and by cement with lime</th>
<th>% Different in $c$ for treated soil by cement and by cement with lime</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated soil</td>
<td>2.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3%cement+2%lime</td>
<td>12</td>
<td>17.86</td>
<td>380</td>
<td>10</td>
<td>26.3</td>
</tr>
<tr>
<td>5%cement+2%lime</td>
<td>13.7</td>
<td>19.64</td>
<td>448</td>
<td>6.35</td>
<td>29.25</td>
</tr>
<tr>
<td>7%cement+2%lime</td>
<td>15.6</td>
<td>23.57</td>
<td>524</td>
<td>8.13</td>
<td>30</td>
</tr>
</tbody>
</table>

3.6. Sixth Group Results

The sixth group included improved sandy soil by mixture 50% of polymer of weight cement added with each percentage of cement, and then treated with 4% of OMC before soaking condition. The relationship between normal stresses versus shear stress is showed in the Figure 17. Practically, the results of shear strength parameters were; the cohesion has a value ranging between (5.5-8) kPa, and the rate value of the angle of internal friction (between untreated soil and treated soil by mixing cement with polymer) between (13.44-20.3%) (See the results in the Table 9).

From the results, the comparison between shear strength treated by cement only and shear strength parameters treated by mixture (cement with polymer) before soaking showed:
Shear strength of soil treated by mixing cement with polymer increased more than the soil treated by cement only. The shear strength parameters increased due to increase the activity between polymer which contains a high percentage of plasticisers and aluminum which work as a cover to the particles of sand with increasing hydration of cement. This reaction was responsible to increase the strength of soil (see also the results in [18] and [21]). The soil treated by 3% of cement with 1.5% of polymer was the optimum value as a result of increase the rate value of shear strength parameters: the cohesion was (37.5%), and the angle of friction was (3.13%), this is more than soil treated by cement only.

It was observed that the shear strength increases with increase mixture content. This phenomenon explained that an increase in polymer and cement content led to enhance bond mechanisms of the sandy soil (see Figures 19 and 20).

![Figure 17. States shear stress versus normal stress for soil sample treated by mixing cement with polymer at 4% of OMC before soaking condition](image)

**Table 9. Shows the Results of Direct Shear Test for Soil Treated by Mixture Cement with Polymer at 4% of OMC before Soaking**

<table>
<thead>
<tr>
<th>Mixed type</th>
<th>Results of the direct shear test</th>
<th>% Increasing in $c$ for treated soil</th>
<th>% Different in $\phi$ for treated soil by cement and by cement with Polymer</th>
<th>% Different in $c$ for treated soil by cement and by cement with Polymer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated soil</td>
<td>$0$</td>
<td>$32$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3%Cement+1.5% Polymer</td>
<td>$5.5$</td>
<td>$36.3$</td>
<td>$13.44$</td>
<td>$3.13$</td>
</tr>
<tr>
<td>5%Cement+2.5% Polymer</td>
<td>$7$</td>
<td>$37.5$</td>
<td>$17.2$</td>
<td>$2.74$</td>
</tr>
<tr>
<td>7%Cement+3.5%Polymer</td>
<td>$8$</td>
<td>$38.5$</td>
<td>$20.3$</td>
<td>$3.22$</td>
</tr>
</tbody>
</table>

**3.7. Seventh Group Results**

The final group included improved sandy soil by mixing 50% of polymer of weight cement added with each percentage of cement, and then treated with 4% of OMC before soaking condition (see Figure 18). The experimental work displayed that; the cohesion has a value range between (2.5-15.6 kPa), and the increasing in the rate value between (380-524%). The angle of internal friction has the rate value of increasing (between untreated soil and treated soil by mixing cement with lime) between (10-15.38%), all results are shown in the Table 10.

To make a comparison between shear strength treated by cement only and shear strength parameters treated by mixture cement with lime after soaking condition, the researcher reached:

- Shear strength of soil treated by mixing cement with polymer increased more than the soil treated by cement only. The shear strength parameters increased as a result of the complete reaction of additives cement plus polymer with particles of soil in the presence of water. The soil treated by 7% of cement with 3.5% of polymer is the optimum value as a result of increasing the rate value of shear strength parameters; the cohesion became (0.94%), and the angle of friction was (30%) (More than the soil treated by cement only).
- The effect of polymer with cement to enhance the shear strength parameters of the sandy soil increased when cement content and polymer increased as a result of increasing pozzolanic reaction of the additives. This led to
the increasing strength of soil, but they decreased as compared with the samples before soaking as it is shown in the Figures 19 and 20. This was also mentioned by [5].

Figure 18. Shows Shear stress versus normal stress for soil sample treated by mixture cement with polymer at 4% of OMC after soaking condition

Figure 19. States the relationship between the percentages of mixing cement with polymer and the angle of internal friction for soil treated at 4% of OMC before and after soaking conditions

Figure 20. Shows the relationship between the percentages of mixing cement with polymer and the cohesion for soil treated at 4% of OMC before and after soaking conditions
Table 10. Shows the Results of Direct Shear Test for Soil Treated by Mixture Cement with Polymer at 4% of OMC after soaking condition

<table>
<thead>
<tr>
<th>Mixed type</th>
<th>Results of the direct shear test</th>
<th>% Increasing in $\phi$ for treated soil</th>
<th>% Increasing in $c$ for treated soil</th>
<th>%Different in $\phi$ for treated soil by cement and by cement with Polymer</th>
<th>%Different in $c$ for treated soil by cement and by cement with polymer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated soil</td>
<td>2.5</td>
<td>28</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3%Cement+1.5%Polymer</td>
<td>12</td>
<td>30.8</td>
<td>10</td>
<td>380</td>
<td>2.67</td>
</tr>
<tr>
<td>5%cement+2.5%Polymer</td>
<td>12.6</td>
<td>32</td>
<td>14.3</td>
<td>404</td>
<td>1.59</td>
</tr>
<tr>
<td>7%cement+3.5%Polymer</td>
<td>15.6</td>
<td>32.3</td>
<td>15.38</td>
<td>524</td>
<td>0.94</td>
</tr>
</tbody>
</table>

### 4. Conclusions

According to the results obtained from the experimental tests. The following results can be summarized:

- Shear strength parameters: the angle of friction ($\phi$), and cohesion ($c$) of sandy soil treated by a mixture of cement with lime, and cement with polymer was increased more than the soil treated by cement only.
- The results of the direct shear test showed that the optimum moisture content (OMC) was 4% that has been obtained from the soil treated by cement with three percentages of water content before soaking condition.
- The high increase of angle of internal friction ($\phi$) is found at rate (23.57%), when treatment of the soil with 7% of cement with 2% of lime after soaking conditions, as compared with the samples before soaking condition. This belongs to complete chemical interaction between the additives and the soil in soaking condition.
- To compare between two mixtures (cement with lime), and (cement with polymer), it was noticed that traditional additives (cement with lime) is still more effective than non-tradition additives (cement with polymer) and also from the cost aspect.

### 5. Conflicts of Interest

The authors declare no conflict of interest.

### 6. References


Comparative Study on Breaking Strength of Burnt Clay Bricks Using Novel Based Completely Randomized Design (CRD)

Zahid Hussain a*, Shamshad Ali a

a Sarhad University of Science and Information Technology, Peshawar, 25000, Pakistan.

Received 09 February 2019; Accepted 10 May 2019

Abstract

The aim of this study is to present the results of breaking strength tests for burnt clay bricks from various historical deposits. The native clay bricks production technique is the known method of brick making, particularly in South Asian countries. Numerous studies have been conducted on hand-molded formed bricks. The clay bricks that were considered for the comparative study, were made from four different clays sources. Their breaking strength was determined using for examining the maximum load at failure and the effects were investigated subsequently. The basic objective of this experimental study was to compare the breaking strength of locally fired clay bricks using a novel based completely randomized design via a single factor with four levels of clay sources representing the factors. For this purpose, 24 brick samples were made from four different clay sources while the breaking strength of each sample was measured. Pairwise comparison trials, including Duncan’s multiple range, Newman–keuls, Fisher’s least and Tukey’s tests were conducted. Based on experimental investigations, the results revealed that using analysis of variance at 95% CI, the difference in breaking strength between clay source of Hyderabad (A) and Rawalpindi (B), followed by Kohat (C) and Peshawar (D) was significant and also the difference among the means of these clay courses was significant which clearly exposed that the clay site and chemical composition has a great impression of the breaking strength of the burnt bricks.

Keywords: Breaking Strength of Burnt Clay Bricks; Experimental Design; Completely Randomized Design (CRD).

1. Introduction

Normally, a good burnt clay brick should be hard enough, well scalded, sound in texture and sharp in outline and measurement and should not break definitely. Burnt clay bricks used in the construction sphere must have certain desired properties that must be achieved [1]. These include density, porosity, breaking strength, thermal stability and fire resistance. Breaking strength is a mechanical property which has assumed greater prominence for various reasons. A greater breaking strength upsurges other properties like resistance to abrasion and flexure, hence this property should be determined more accurately [2]. Breaking strength relies on the chemical composition of clay used, the manufacturing techniques followed by the physical shape and dimension of the brick.

The crushing resistance differs from 3.5 N/mm² for normal soft facing bricks to 140 N/mm² for engineering clay bricks in case tested in the dry state. In general, breaking strength declines with increasing porosity, however strength is also affected by composition of clay and firing temperate [3]. As per mechanical characteristics regarding solid bricks materials as shown in Table 1, the bricks formed by hand as shown in Figure1 will have comparatively inferior quality, especially breaking strength, and will incline to have uneven dimensions. Bricks prepared in this way have been used in

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http://dx.doi.org/10.28991/cej-2019-03091320

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structures which have lasted for centuries [4]. Their permanency has depended on the excellence of the constituents, the skill of the handicraft worker and the environment in which they were used [5].

Table 1. Bricks and clay mechanical characteristics (Breaking Strength)

<table>
<thead>
<tr>
<th>Unit Type</th>
<th>Breaking strength, psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forming method</td>
<td></td>
</tr>
<tr>
<td>Moulded</td>
<td>5293 (36.5)</td>
</tr>
<tr>
<td>Extruded</td>
<td>11305 (77.9)</td>
</tr>
<tr>
<td>Solid brick</td>
<td></td>
</tr>
<tr>
<td>Fire Clay</td>
<td>15343 (106.8)</td>
</tr>
<tr>
<td>Raw material</td>
<td></td>
</tr>
<tr>
<td>Shale</td>
<td>11258 (77.6)</td>
</tr>
<tr>
<td>Others</td>
<td>9159 (63.2)</td>
</tr>
</tbody>
</table>

The primary perilous element for producing burnt clay bricks is the determination of a good quality of clay. The composition of clay differs a wide range of alumina, iron, lime, sulfur, silica sand and some amount of phosphate. During the process of firing the temperature must remain constant to get the best quality product [6]. Obviously it will develop additional issues such as low breakage strength and increase rejection rate. When stresses present within a drying brick, cracks can appear to relieve them [7]. These stresses are developed when a brick does not dry stability. Therefore, when the later part needs to shrink, a sudden crack may appear to dismiss the stress. Minor drying cracks may normally develop during the process of firing, particularly if noteworthy firing shrinkage happens [8]. Cracking and defect problems occur due to plasticity of clay, pressing pressure, drying process time, kiln temperature [9]. Such a sample is shown in Figure 2. Although there could be certain questions, however assuming that the lower level of breaking strength is associated with the inadequate quality of clay used. Hence this research study is conducted to determine the best source of clay that can result in better output. To investigate and analyze the potential factors influencing on the breaking strength of the bricks, a novel based completely randomized design with single factor experiment has been designed to take additional annoyance aspects into account.

2. Completely Randomized Design

This approach is best suited for analyzing the effect of a single factor to consider other annoyance factor into account [10]. It basically associates the tenets of a specific response variable with other necessary levels while the levels of factors designed randomly [11]. The design strategy is based on three key factors, \( f \) = number of factors which is usually taken as 1 followed by \( L \) = number of possible levels and \( N \) = replications numbers. Starting design phase is to set the hypothesis using ANOVA making the objective to test the right hypotheses around the means of treatment and to
approximate them[12]. If \( X_{ij} \) is the jth response observation for specific factor treatment level \( i \); \( N \) observations, \( T_i \) observations total, \( X_i \). As the mean for treatment level, \( T \) the grand total and \( \bar{X} \) the grand mean, then classic data of design of experiment for a single factor may look as shown Table 2.

\[
T_i = \sum_{j=1}^{n} X_{ij} \tag{1}
\]

\[
\bar{X}_i = \frac{\sum_{j=1}^{n} X_{ij}}{n} \tag{2}
\]

<table>
<thead>
<tr>
<th>Table 2. Data orientation for single factor design of experiment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor level</td>
</tr>
<tr>
<td>---------------</td>
</tr>
<tr>
<td>1.</td>
</tr>
<tr>
<td>2.</td>
</tr>
<tr>
<td>3.</td>
</tr>
<tr>
<td>a.</td>
</tr>
<tr>
<td>Total</td>
</tr>
</tbody>
</table>

2.1. Analysis of Variance

Considering the necessary parameters for segregating of total variability present in specific component parts that is:

\[
SS_{Total} = SS_{due to treatments} + SS_{due to error}:
\]

\[
SS_{Total} = \sum_{i=1}^{a} \sum_{j=1}^{n} (X_{ij} - \bar{X})^2 \tag{3}
\]

Making appropriate hypothesis such as: Null Hypotheses (H0): \( T1 = T2 = T3 \ldots Tn = 0 \) and Alternative Hypotheses (H1) \( Ti \neq 0 \), at least for any one observation, then setting a 5% significance level (\( \alpha = 0.05 \)) for conducting ANOVA, if the test value (\( F_0 \)) greater than critical value (\( Fc \)) based on levels of factors and degree of freedom then it is decided to reject null hypothesis (H0) while the ANOVA computations results are presented using Table 3.

<table>
<thead>
<tr>
<th>Table 3. ANOVA for single factor design of experiment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Source</td>
</tr>
<tr>
<td>Treatments</td>
</tr>
<tr>
<td>Errors</td>
</tr>
<tr>
<td>Total</td>
</tr>
</tbody>
</table>

2.2. Estimation of Model Parameters and Residuals

Typically, a single factorial design of experiment model is expressed as:

\[
X_{ij} = \mu + T_i + e_{ij} \tag{4}
\]

Showing that the total mean is estimated by the grand average and the effect of treatment should be equal to the variance among the treatment and grand means[13].

\[
e_{ij} = X_{ij} - \bar{X}_i \tag{5}
\]

However, if these conventions are valid somehow then the ANOVA test is valid which is analysed by the observations and treatments residuals (\( R_0 \)).

\[
R_{ij} = X_{ij} - \bar{X}_{ij} \text{ (error)} \tag{6}
\]

Reconsidering Equation 4 and expanding Equation 6:

\[
R_{ij} = X_{ij} - \bar{X}_i \tag{7}
\]

It means that estimation of any remark in the \( i^{th} \) usage is the consistent mean of treatment [14]. Conversely, If the current model is acceptable, then all the residuals must be structure fewer and analysed graphically [15].

2.3. Model Acceptability Checking

Normality check: Recognizes the process errors that must be distributed normally while the residuals plot for normal probability distribution would look like a straight line for the hypothesis to be valid [16]. Reasonable partings from this straight line may not be thoughtful because the sample extent might be trivial with no availability of outliers [17].
Independence check: If the scheme of errors pertain to design in data order determine an arbitrary strew, it will straight forward to identify the errors are self-determining [18]. If any correlation happens among the residuals it will show the defilement of the independent supposition [19]. However if the corresponding residuals are located randomly then it will show that they are autonomous [20].

Constant variance check: If the model is acceptable, the plot of residuals should be structure less and should not be related to any variable including the predicted response [21]. The plots of residuals versus predicted response should appear as a parallel band centered about zero. If the spread of residuals increases, it will display that the error variance growths with the mean [22]. A valid model validation will show that no unusual structure is present that indicate that the model is correct [23].

2.4. Analysis of Treatments Mean

If the ANOVA result is to reject the null hypothesis that is, there exist variances among the means of the treatment but which means differ is not known. Hence, it is desirable to make comparison between the means of treatment [24]. This is also convenient to classify the finest and favored treatments for possible use in repetition. The best conduct is the one that resembles to the mean of treatment that optimizes the process response [25]. Other favored treatments of mean may be recognized over multiple comparison approaches and conducting ANOVA is performed [26]. They include:

Duncan’s multiple range test: The test is suitable to compare the means of all pairs using a definite sequence.

*Step 1:* Arranging the means of treatment means in ascending form;

*Step 2:* Calculating the standard errors (Sx) of means (SX):

\[
S_x = \sqrt{\frac{MS_e}{N}} \quad (8)
\]

*Step 3:* Computing the values of r (p, f) for p = 2, 3… a, from Duncan’s multiple ranges considering a as significance level and f as the degrees of freedom of error;

*Step 4:* Compute least significant ranges (Rp) for p = 2, 3… a;

\[
R_p = (S_x) r_a(p,f) \quad (9)
\]

*Step 5:* Test the observed differences between the means against the least significant ranges (Rp) as follows;

**Cycle 1:** Compare the difference between largest and smallest mean with Ra. Compare the difference between largest mean next smallest mean with Ra – 1 until all comparisons with largest mean [27].

**Cycle 2:** Test the variance between the second major and the minor mean with Ra – 1 until all possible pairs of means a (a-1)/2 are tested.

Inference: If the observed difference between any two means exceed the least significant range, the difference is considered as significant [28].

Newman – Keuls test: Normally, the process of this test is similar to that of Duncan’s multiple range test excluding that studentized ranges are used [29]. That is, in Step 4 of Duncan’s procedure Rp is replaced by Kp, where q (p, f) values are obtained from studentized range table.

\[
K_p = q_a(p,f) \quad (10)
\]

Fisher’s Least Significant Difference (LSD) test: In this test means of all possible pairs are compared with LSD, where

\[
LSD = t_{a/2,N} - A \sqrt{\frac{1}{N_i} + \frac{1}{N_j}} \quad (11)
\]

If the absolute variance between any two means surpasses LSD, then two means are considered as significantly different [30]. Using a balanced design, n1 = n2 = … = n. hence,

\[
LSD = t_{a/2,N} - A \sqrt{\frac{2MS_e}{N}} \quad (12)
\]

Tukey’s test: In this test studentized range statistic is used and the possible pairs of means are then compared with Ta (Risch, 1981). If the absolute alteration between any two means surpass Ta, then it is concluded that the two means differ significantly [31].
\[ T_\alpha = q_\alpha(a, f) \sqrt{\frac{2MS_0}{N}} \]  

### 2.5. Comparison of Means Using Contrasts Approach

A contrast presents a linear grouping of treatment totals and the addition of its coefficients may be equal to 0. A simple comparison of means with contrasts is equation based, while the sum of square of an individual contract is determine using expression [32].

\[ C_1 = T_1 - T_2 \]  

\[ SS_C = \frac{\sum_{i=1}^{a} C_i T_i}{N \sum_{i=1}^{a} C_i} = \frac{C_i^2}{N \sum_{i=1}^{a} C_i} \]  

Here \( C_i \) represents the contrast of totals. However if \( F_0 > F_{\alpha, 1, N-A} \) then the decision is to reject the null hypotheses (H\(_0\)) [33].

**Orthogonal Contrast:** In any contrast if one of the treatment level is absent, its corresponding coefficient will be 0 that is \( C_1 = T_1 + T_2 \) and \( C_2 = T_3 + T_4 \).

### 3. Methodology

For better and easy understand of the research methodology workflow and finding out which step is excessive and which improvement should be initiated, the experimental work was conducted in accordance to the research methodology process flowchart as shown in Figure 3. This flowchart will assure visual clarity, instant communication, effective coordination, effective analysis and a better decision making. The steps are described in sequence.

**Figure 3. Flowchart steps for conducting research work**

During the initial phase, four different samples of clays were acquired from four different cities of Pakistan, namely Hyderabad (A), Rawalpindi (B), Kohat (C) and Peshawar (D) as presented in Figure 4. It is intended to govern the unsurpassed source of clay that results in comparatively enhanced breaking strength. During the next phase a single-factor design of experiment was designed in order to consider the four clay sources as the four levels of the factor. Later, six different samples of bricks were made from each clay source. Finally, the corresponding breaking strength was measured. Using Equations 1 and 2 the mean breaking strength of each source has determined and result values are shown in Table 4. Figure 5 describes the graphical comparison of breaking strength of four clay sources while Figure 6 shows empirical cumulative distribution function CDF Plot to assess the fit of distributed data values of each individual observation contrary to the percentage of the values that are smaller or equal to that specific value. The plot examines the distribution of breaking strength of each single brick made from different clay source. It has well observed from the graph that the stepped line follows the fitted distribution line thoroughly, it means that the data fits the distribution well.

**Figure 4. Clay samples of the four sites**
Table 4. Breaking strength of burnt bricks (Kg/cm$^2$)

<table>
<thead>
<tr>
<th>Source</th>
<th>Observations</th>
<th>Ti</th>
<th>Xi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyderabad (A)</td>
<td>98 91 96 96 100 103</td>
<td>584</td>
<td>97.33</td>
</tr>
<tr>
<td>Rawalpindi (B)</td>
<td>84 86 89 90 86 88</td>
<td>523</td>
<td>87.17</td>
</tr>
<tr>
<td>Kohat (C)</td>
<td>90 91 94 86 88 92</td>
<td>541</td>
<td>90.17</td>
</tr>
<tr>
<td>Peshawar (D)</td>
<td>77 81 84 80 78 85</td>
<td>485</td>
<td>80.83</td>
</tr>
</tbody>
</table>

|               |               |     |     |
|               |   2133        | 355.50 |

4. Results and Discussions

4.1. Data analysis (ANOVA)

The calculations required for ANOVA are computed on the basis of Equations 1, 2 and 3 while other supplementary variables along with the hypothesis tested in this case are: $H_0: T_1 = T_2 = T_3 \ldots T_n = 0$ that is there is no significant different between the mean of breaking strength of four cities clay source. $H_1: T_i \neq 0$, that is mean of breaking strength of four cities clay source differ significantly. At 5% significance level ($\alpha = 0.05$) and degree of freedom as 3 and 20 for
the given data and using the proportion arguments of the F distribution (F0.05) value, the critical value (F critical) = 3.10 which clearly shows that F0 > F (crit) that is 28.18 > 3.098 hence it is decided to discard the null hypothesis (H0) and admit the alternative hypothesis (H1) [34]. Therefore, it is concluded that mean of breaking strength of four clay source differ significantly. ANOVA test results are presented in Table 5.

Table 5. Summary of ANOVA

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>SS</th>
<th>DF</th>
<th>MS</th>
<th>F</th>
<th>P</th>
<th>F(c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Between Groups</td>
<td>844.79</td>
<td>3</td>
<td>281.60</td>
<td>28.18</td>
<td>2.2E-07</td>
<td>3.0</td>
</tr>
<tr>
<td>Within Groups</td>
<td>199.83</td>
<td>20</td>
<td>9.99</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>1044.6</td>
<td>23</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Graphical presentation of results using Minitab are revealed in Figure 7 that shows the F-distribution value. Here shaded part characterizes the likelihood of perceiving the F-distribution value which is truly larger than the F-value that has been computed and obtained. F-value of 28.18 fall inside the rejection region. This prospect is high sufficient to castoff the null hypothesis at significance level of 0.05. Hence it is decided that not all the clay sources means are identical. Similarly, Figure 8 shows normal probability plot of the residuals of samples distribution which were computed using Equations 4 to 7. It represents the residuals against their predictable values when they are normally distributed. Since the plot of the residuals are arranged in such a way that they are making nearly a straight line, it thus confirm the hypothesis that the residuals are ordinarily dispersed while Figure 9 highlights the residuals plot versus fitted values of samples distributions [35]. Here the residuals are located on the vertical axis while the correspondent fitted values on the horizontal axis respectively. Since, preferably, the points ought to fall randomly on both sides of 0 and should have no perceptible patterns in the points. Therefore, it clearly confirms the hypothesis that the samples mean values residuals are randomly distributed with continuous variance [36].

Figure 7. F distribution plot of means significances at (α = 0.05)

Figure 8. Normal Probability for residuals of samples distribution
4.2. Duncan’s Multiple Range Results

Step 1. Arranging the treatment means in ascending order:

<table>
<thead>
<tr>
<th>Source</th>
<th>Peshawar (D)</th>
<th>Rawalpindi (B)</th>
<th>Kohat (C)</th>
<th>Hyderabad (A)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>80.83</td>
<td>87.17</td>
<td>90.17</td>
<td>97.33</td>
</tr>
</tbody>
</table>

Step 2. Calculating the standard error (s_e) means using Equation 8 which yields 1.29

Step 3. Obtaining values of r (p, f) for p = 2, 3, and 4 from significant ranges for Duncan’s multiple ranges test for \( r_{0.05} (p, f) \) is:

\[
r_{0.05} (p, 20) \quad 2 \quad 3 \quad 4
\]

<table>
<thead>
<tr>
<th></th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r_{0.05} )</td>
<td>2.95</td>
<td>3.1</td>
<td>3.18</td>
</tr>
</tbody>
</table>

Step 4. Computing least significant ranges (LSR) = \( Rp \) for \( p = 2, 3 \) and 4 are calculated using Equation 9:

<table>
<thead>
<tr>
<th>LSR</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.807</td>
<td>4.000</td>
<td>4.103</td>
</tr>
</tbody>
</table>

Step 5. Comparison of pairs mean and the absolute difference with LSR.

**Cycle 1**

<table>
<thead>
<tr>
<th>Pair</th>
<th>LSR</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
<th>Significant</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyderabad (A) vs Peshawar (D)</td>
<td>16.5</td>
<td>&gt; 4.103</td>
<td></td>
<td></td>
<td>Significant</td>
</tr>
<tr>
<td>Hyderabad (A) vs Rawalpindi (B)</td>
<td>10.16</td>
<td>&gt; 4.000</td>
<td></td>
<td></td>
<td>Significant</td>
</tr>
<tr>
<td>Hyderabad (A) vs Kohat (C)</td>
<td>7.16</td>
<td>&gt; 3.807</td>
<td></td>
<td></td>
<td>Significant</td>
</tr>
</tbody>
</table>

**Cycle 2**

<table>
<thead>
<tr>
<th>Pair</th>
<th>LSR</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
<th>Significant</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kohat (C) vs Peshawar (D)</td>
<td>9.34</td>
<td>&gt; 4.000</td>
<td></td>
<td></td>
<td>Significant</td>
</tr>
<tr>
<td>Kohat (C) vs Rawalpindi (B)</td>
<td>3.00</td>
<td>&lt; 3.807</td>
<td></td>
<td></td>
<td>Not Significant</td>
</tr>
</tbody>
</table>

**Cycle 3**

<table>
<thead>
<tr>
<th>Pair</th>
<th>LSR</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
<th>Significant</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rawalpindi (B) vs Peshawar (D)</td>
<td>6.34</td>
<td>&gt; 3.807</td>
<td></td>
<td></td>
<td>Significant</td>
</tr>
</tbody>
</table>

Thus it can be evidently judged that there exists not a significant difference between clay source of Rawalpindi (B) and Kohat (C) while mean of Hyderabad (A) and Peshawar (D) differ significantly from B and C. Figure 10 shows Dunnet test control mean of the clay source. These are grouped as follows:

<table>
<thead>
<tr>
<th>Clay Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyderabad (A)</td>
</tr>
<tr>
<td>Rawalpindi (B) &amp; Kohat (C)</td>
</tr>
<tr>
<td>Peshawar (D)</td>
</tr>
</tbody>
</table>
4.3. Newman – Keuls Test Results

**Step 1 and 2.** are calculated using Equation 10 for Duncan’s multiple range test results.

**Step 3.** Obtaining values of \( q(p, f) \) for \( p = 2, 3, \) and \( 4 \) from significant ranges for Newman – Keuls ranges, test using equation (10) for \( q_{0.05}(p, f) \) is.

\[
\begin{array}{c|c|c|c}
q_{0.05}(p, 20) & 2 & 3 & 4 \\
2.95 & 3.58 & 3.96 \\
\end{array}
\]

**Step 4.** Computing least significant ranges (LSR)\( = (R_p) \) for \( p = 2, 3 \) and \( 4 \) and the three Least Significant Ranges are:

<table>
<thead>
<tr>
<th>LSR</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.807</td>
<td>4.619</td>
<td>5.110</td>
</tr>
</tbody>
</table>

**Step 5.** Comparison of pairs mean and the absolute difference with LSR.

<table>
<thead>
<tr>
<th>Cycle 1</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyderabad (A) vs Peshawar (D)</td>
<td>16.5 &gt; 5.110 Significant</td>
</tr>
<tr>
<td>Hyderabad (A) vs Rawalpindi (B)</td>
<td>10.16 &gt; 4.619 Significant</td>
</tr>
<tr>
<td>Hyderabad (A) vs Kohat (C)</td>
<td>7.16 &gt; 3.807 Significant</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cycle 2</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Kohat (C) vs Peshawar (D)</td>
<td>9.34 &gt; 4.619 Significant</td>
</tr>
<tr>
<td>Kohat (C) vs Rawalpindi (B)</td>
<td>3.00 &lt; 3.807 Not Significant</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cycle 3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Rawalpindi (B) vs Peshawar (D)</td>
<td>6.34 &gt; 3.807 Significant</td>
</tr>
</tbody>
</table>

Hence, the final results of Newman – Keuls test are similar to Duncan’s multiple range test showing that there exists not a significant difference between clay source of Rawalpindi (B) and Kohat (C) while mean of Hyderabad (A) and Peshawar (D) differ significantly from B and C. These are grouped as follows:

| Hyderabad (A) | Rawalpindi (B) & Kohat (C) | Peshawar (D) |

4.4. Fisher’s Least Significant Difference (LSD) Test Results

Since the absolute variance between any two means surpasses LSD hence Equations 11 and 12 are used to determine the value of LSD is equal to 3.81. Comparing the sources for any significance difference: Hyderabad Source (A): \( = X_1 = 97.33 \); Rawalpindi Source (B): \( = X_2 = 87.17 \); Kohat Source (C): \( = X_3 = 90.17 \); Peshawar Source (D): \( = X_4 = 80.83 \) and comparison of means of all possible pairs is carried out as follows.
<table>
<thead>
<tr>
<th>Source</th>
<th>Mean</th>
<th>Absolute Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyderabad (A)</td>
<td>97.33</td>
<td></td>
</tr>
<tr>
<td>Rawalpindi (B)</td>
<td>87.17</td>
<td></td>
</tr>
<tr>
<td>Kohat (C)</td>
<td>90.17</td>
<td></td>
</tr>
<tr>
<td>Peshawar (D)</td>
<td>80.83</td>
<td></td>
</tr>
</tbody>
</table>

The absolute dissimilarity in the two means for judgement:

<table>
<thead>
<tr>
<th>Comparison</th>
<th>Absolute Difference</th>
<th>p-value</th>
<th>Conclusion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyderabad (A) vs Rawalpindi (B)</td>
<td>10.16</td>
<td>&gt; 3.81</td>
<td>Significant</td>
</tr>
<tr>
<td>Hyderabad (A) vs Kohat (C)</td>
<td>7.16</td>
<td>&gt; 3.81</td>
<td>Significant</td>
</tr>
<tr>
<td>Hyderabad (A) vs Peshawar (D)</td>
<td>16.5</td>
<td>&gt; 3.807</td>
<td>Significant</td>
</tr>
<tr>
<td>Rawalpindi (B) vs Kohat (C)</td>
<td>-3</td>
<td>&gt; 3.65</td>
<td>Not Significant</td>
</tr>
<tr>
<td>Rawalpindi (B) vs Peshawar (D)</td>
<td>6.34</td>
<td>&gt; 3.81</td>
<td>Significant</td>
</tr>
<tr>
<td>Kohat (C) vs Peshawar (D)</td>
<td>9.34</td>
<td>&gt; 3.81</td>
<td>Significant</td>
</tr>
</tbody>
</table>

Similarly, this test also shows the same results describing that there exists not a significant difference between clay source of Rawalpindi (B) and Kohat (C) while mean of Hyderabad (A) and Peshawar (D) differ significantly from B and C. Figure 11 shows Fisher’s Least Significant Difference (LSD) test results of the clay source. These are grouped as follows:

| Source          | |                   |                   |
|----------------|---------------------|
| Hyderabad (A)  | Rawalpindi (B) & Kohat (C) | Peshawar (D) |

4.5. Tukey’s Test

Equation 13 is used to determine the value of LSD is equal to 5.11 and using the same strategy for comparing the sources for any significance difference as conducted in Fisher’s Least Significant Difference (LSD) test, observing the absolute dissimilarity in the two means for judgement:

<table>
<thead>
<tr>
<th>Comparison</th>
<th>Absolute Difference</th>
<th>p-value</th>
<th>Conclusion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyderabad (A) vs Rawalpindi (B)</td>
<td>10.16</td>
<td>&gt; 5.11</td>
<td>Significant</td>
</tr>
<tr>
<td>Hyderabad (A) vs Kohat (C)</td>
<td>7.16</td>
<td>&gt; 5.11</td>
<td>Significant</td>
</tr>
<tr>
<td>Hyderabad (A) vs Peshawar (D)</td>
<td>16.5</td>
<td>&gt; 5.11</td>
<td>Significant</td>
</tr>
<tr>
<td>Rawalpindi (B) vs Kohat (C)</td>
<td>-3</td>
<td>&lt; 5.11</td>
<td>Not Significant</td>
</tr>
<tr>
<td>Rawalpindi (B) vs Peshawar (D)</td>
<td>6.34</td>
<td>&gt; 5.11</td>
<td>Significant</td>
</tr>
<tr>
<td>Kohat (C) vs Peshawar (D)</td>
<td>9.34</td>
<td>&gt; 5.11</td>
<td>Significant</td>
</tr>
</tbody>
</table>

Finally, this test also produces the same results describing that there exists not a significant difference between clay source of Rawalpindi (B) and Kohat (C) while mean of Hyderabad (A) and Peshawar (D) differ significantly from B and C. These are grouped as follows:

| Source          | |                   |                   |
|----------------|---------------------|
| Hyderabad (A)  | Rawalpindi (B) & Kohat (C) | Peshawar (D) |
4.6. Contrasts Results

The results are based on Equations 14 and 15 respectively. Since there are four levels that four sources of clays used, thus three orthogonal contrasts may be formed easily.

\[
\begin{align*}
C_1 & : T_1 - T_2 & : 61 \\
C_2 & : T_1 + T_2 - T_3 - T_4 & : 81 \\
C_3 & : T_3 - T_4 & : 56
\end{align*}
\]

Consequently, the sum of the contrast SS should be equal to SS\(_T\) on the basis of apportioned into three (A – 1) with a single degree of freedom. These results are summarized in Table 6. At F 5%, 1, 20 = 4.35. At 5% significance level (\(\alpha = 0.05\)), all the three contrasts are significant. Thus it is concluded that the difference in breaking strength between clay source of Hyderabad (A) and Rawalpindi (B), and Kohat (C) and Peshawar (D) is observed to be significant and also the difference among the means of these clay courses is significant.

Table 6. Summary of ANOVA summary for testing contrasts

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>SS</th>
<th>DF</th>
<th>MS</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Between treatments</td>
<td>844.79</td>
<td>3</td>
<td>281.60</td>
<td>28.18</td>
</tr>
<tr>
<td>C1</td>
<td>310.083</td>
<td>1</td>
<td>310.08</td>
<td>28.67</td>
</tr>
<tr>
<td>C2</td>
<td>273.38</td>
<td>1</td>
<td>273.38</td>
<td>28.43</td>
</tr>
<tr>
<td>C3</td>
<td>261.33</td>
<td>1</td>
<td>261.33</td>
<td>28.51</td>
</tr>
<tr>
<td>Error</td>
<td>199.84</td>
<td>20</td>
<td>9.99</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>1044.63</td>
<td>23</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5. Conclusion

In this study it was assumed that the lower level of breaking strength is due to the poor quality of clay used. The innovation of the current work is the inclusion, experimental and numerical based investigation using completely randomized design approach. For the purpose of investigation four different clay samples were collected from four different cities, namely Hyderabad, Rawalpindi, Kohat and Peshawar. 24 brick samples were made from these four different clays sources. The breaking strength of each specimen was measured and considered for exploring any significance difference between the clay sources. In order to check the contrast of each clay effect, multiple comparisons of means using contrasts approach has also been conducted. Based on experimental investigations using completely randomized design (CRD) concerning breaking strength of the burnt clay bricks, the results may be summarized that using analysis of variance at 95% CI, the value of Fisher test at 5%, is 1, 20 = 4.35, hence at 5% significance level (\(\alpha = 0.05\)), all the three contrasts were significant. Hence, the study revealed that the difference in breaking strength between clay source of Hyderabad (A) and Rawalpindi (B), followed by Kohat (C) and Peshawar (D) was significant and also the difference among the means of these clay courses was significant. Which clearly proved that the clay location and compositions have a vital role and a great impression of the breaking strength of the burnt bricks.

6. Acknowledgements

Current work has been facilitated by Sarhad University of Science and Information Technology, Peshawar Pakistan. We are really grateful to our supporting colleagues from Voronezh State University, Russian Federation who provided awareness and proficiency that helped in completing the research.

7. Funding

This research work was supported by Sarhad University of Science and Information Technology, Peshawar Pakistan.

8. Conflict of Interests

The authors declare no conflict of interest.
9. References


Research of Scale Inhibitors in Downhole Equipment

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Received 02 March 2019; Accepted 20 May 2019

Abstract

The article is devoted to the problem of salt deposition in the West-Agan field of Western Siberia in various types of oil equipment during the extraction and transportation of petroleum products, as well as research and identification of the most effective inhibitors. The article is devoted to the problem of salt deposits in various types of oil equipment during the extraction and transportation of petroleum products. Salinity (scale) adversely affects the surface of pipes, the working parts of pumps, installations in contact with formation waters. When this occurs, the parts are jammed or break. The article describes methods for solving the problem of scaling using inhibitors. The possibility of SIs supplying in the reservoir with the aim of its subsequent removal in the composition of the reservoir fluid [1-4]. The data on the adsorption-desorption capacity of scale inhibitors on core samples were studied. Adsorption was tested under dynamic conditions on the FDS-210 filtration unit, filled with a core from the West Agan field. The core was saturated with a scale inhibitor through pumping a 1% solution through it. Desorption of inhibitors was modeled by passing the bottom-water model through the cell under the same pumping conditions. The results of processing the experimental data are obtained; the figures are shown in the form of the dependence of the Freundlich equation relating the magnitude of adsorption to the current concentration of inhibitors, as well as the profile of the removal of Descum-2 scale inhibitors H-3611-Descum-4 and OEDPK. The paper presents the results of modeling the removal of Descum-2 inhibitors of the mark H-3611-A, Descum-4 of the mark S and OEDFC in the form of a table. The use of the studied inhibitors on the West Agan field allows to increase the duration of the maintenance-free period of the well equipment.

Keywords: Adsorption-Desorption Capacity; Core; Inhibitors; Field Test; Productive Stratum; Scaling.

1. Introduction

Complications associated with the development of oil and gas wells, are an aggressive component of inorganic salts of various types. These deposits disable ground valves, tubing, electrical centrifugal and piston pumps, production string and field equipment [5]. These phenomena reduce the efficiency of well pumps, lead to a decrease in the rise of oil and, consequently, to a decrease in well production [6].

This article discusses the results of laboratory studies of scale inhibitors Descum-2 of mark H-3611-A and Descum-4 of mark S for deposits of the Khanty-Mansi Autonomous District. Determination of the adsorption-desorption capacity of inhibitors on core samples of the West Agan field was carried out to assess the ability of inhibitors to effectively protect downhole equipment from scaling when setting the reagent solution into the reservoir. The choice of the reservoir core is due to the need for analysis in the most difficult conditions of fluid passing through the rock. The tests were
carried out according to the following methodological and regulatory documents:

- Agreement of the company OJSC “NK “Rosneft”” No. III-01.05P-0339 “Procedure for the use of chemical reagents at hydrocarbon production facilities”.
- Terms of Reference for laboratory testing of scale inhibitors in producing wells of fields for injection by the method of injection into the reservoir.

Laboratory tests of Descum-2 scale inhibitors of mark H-3611-A and Descum-4 mark S were carried out in the accredited laboratory of OJSC NizhnevartovskNIPIneft (Accreditation certificate ROSS RU00001.21NK28).

The customer handed over samples of scale inhibitors Descum-2 of mark H-3611-A, Descum-4 of mark S with a volume of 1 liter each, manufactured in accordance with TU 20.59.42-126-94296805-2017 and TU 20.59.42-127-94296805-2017, documentation on these inhibitors and the terms of reference for laboratory testing.

The innovation of this article is the laboratory testing of the above inhibitors and the research of the reagent in order to comply with the requirements of the regulatory documents of Rosneft for the application of pilot tests for the protection of borehole equipment from scaling.

### 2. Research Methodology

In the process of studying the problem of scaling is the use of scale inhibitors (ISO). Salt deposits in the oil and gas sector have a detrimental effect on oil production, which is a very difficult problem in the development and exploitation of oil fields. Huge money is spent to restore and replace oilfield equipment due to scaling every year. In order to combat salt formation, the oil workers use inhibitor protection.

Many of the world’s scientists are tackling this problem: Mike Crabtree from Aberdeen, Scotland, David Eslinger from Tusla, Oklahoma, USA, Phil Fletcher and Matt Miller from Sugar Land, Texas USA, Ashley Johnson from Rosarhon, Texas, George King from BP Amoco Corporation Houston, Texas and many others. Russian scientists are also studying this problem: Korobeinikova DS, Sibiryakov KA, Tarkhov L.G. (Perm), A.N. Semenov, D.V. Markelov (LLC Yuganskneftegaz), V.V. Ragulin, A.I. Voloshin, A.G. Mikhailov (YuNG-NTTs Ufa LLC), Kanzafarov F.Ya. (LLC NizhnevartovskNIPIneft). After analyzing the work of the authors, we concluded that the use of scale inhibitors, such as Descum-2 mark H-3611-A and Descum-4 mark S for the fields of the Khanty-Mansi Autonomous Okrug were not investigated.

This paper is the result of works on laboratory modeling of scale inhibitors, such as Descum–2 mark H-3611-A and Descum–4 mark S for fields of the Khanty-Mansi Autonomous Okrug. The possibility of supplying scale inhibitors to the reservoir for the purpose of its subsequent removal as part of the formation fluid was considered in the course of our study.

The studies were carried out on a filtration unit FDS-210, filled with the core of the West Aganskoye field using the following algorithm. Initially, inhibitors were fed into the core holder in saline water with disintegrated rock. The filtration rate of fluid through the rock was 1.5 m / day. The core was saturated with a scale inhibitor by pumping a 1% inhibitor solution through the core. At the output of the core holder, 3 ml solution was selected, which was analyzed for the content of the active substance of the commercial form of scale inhibitors. To determine the content of phosphonates in solutions, a standard technique was applied (the photometric method for determining the concentration of phosphorus-containing inhibitors of salt formation in saline water is based on the reaction of the interaction of phosphate ions obtained from phosphonate with molybdate ion in an acidic medium). The concentration of inhibitors was determined by calibration straight. After passing 10 pore volumes through the porous medium and reaching the concentration of inhibitors in the outgoing solution corresponding to the initial concentration, the dosing of the inhibitors was stopped, and the core with the inhibitor was kept for 2 hours to adsorb the reagent on the rock.

Desorption of inhibitors was carried out by produced commercial water through the cell. Pumping modes have not changed. At the outlet of the column, 3 ml of the working solution is selected, which was analyzed for the content of scale inhibitors. After passing about 30 pore volumes and reaching the concentration of the active substance in the solution corresponding to the detection threshold (~ 1 mg/l), the experiment was stopped. The obtained inhibitor removal curves were processed using the Squeeze V software package, the "ADSORPTION ISOTHERM DERIVATION MODEL" subroutine. The initial data for this subprogram are the results of the removal of scaling inhibitors: the dependence of the concentration on the volume of the pumped liquid, expressed both in absolute units and in the number of pore volumes.

The results of processing the data obtained are shown graphically in Figure 1 as a dependence of the Freundlich equation relating the magnitude of adsorption to the current concentration of inhibitors. In the course of work, methods of laboratory research, analytical character were used; software (Squeeze V program) was used by applying test simulation of the process.
In the process of oil production in various elements of the oilfield system, inorganic salts may be deposited from the formation waters [7]. Salt formation is noted in wellhead equipment, discharge lines of wells, metering installations; however, most often this process takes place in production wells, which leads to deposition on working parts and surfaces of ESP, as well as other hydraulic machines, dense, stone sediment of 0.5-1.0 mm thick [8].

The heat exchange is disturbed because of sedimentation, the impellers of the pump unit become jammed and the shaft breaks [9]. Such complications inevitably lead to the appearance of premature failures of the underground equipment of pumping wells; as a result, there is a decrease in the work of the well during the turnaround time, and therefore a drop in oil production occurs [10].

In the fields of Western Siberia, calcium carbonate is the main component of solid deposits. In addition to CaCO$_3$, silica (SiO$_2$), corrosion products (FeCO$_3$), and hydrocarbons are found in the composition of the deposits [11].

In oil field practice, one of the main methods of controlling salt deposits is the use of scale inhibitors. Various technologies are used to supply scale inhibitors to production wells. In this paper, we consider the possibility of supplying scale inhibitors to the reservoir with a view to its subsequent removal as part of the formation fluid. To implement this technology, data on the adsorption-desorption ability of scale inhibitors on core samples are needed [12].

In the study of scale inhibitors of downhole equipment, adsorption under dynamic conditions was studied on a filtration unit FDS-210, filled with a core of the West Agan field. Solutions of inhibitors in saline water were fed to the core holder with disintegrated rock. Cell size 30x30. The linear filtration rate of the fluid through the rock was 1.5 m/day.

In the study of desorption of inhibitors, produced water was passed through the model cell [13], under the same pumping regimes. The solution at the outlet of the cell was analyzed for the content of scale inhibitors. The results of the experiment were processed using the Squeeze V software complex and obtained in the form of curves of inhibitor removal lines. The study of adsorption-desorption properties was carried out for scale inhibitors under the trade names Descum-2 and Descum-4, using the HEDP inhibitor as the base of comparison.

The results of processing the data obtained are shown in Figure 1 as a dependence of the Freundlich equation relating the magnitude of adsorption to the current concentration of inhibitors [14].

![Figure 1. Adsorption isotherm of Descum-2 inhibitors of mark H-3611-A, Descum-4 of mark S on the core rocks of the West Agan deposit](image-url)

According to the testing of the adsorption-desorption properties of 10% solutions of salt-inhibitors Descum-2 mark H-3611-A, Descum-4 marks S and 10% solution of scale inhibitors HEDP for comparison, test simulations of the squeeze were performed (Squeeze V program) with given volumes the main commissioning of 10 m 10% solution of inhibitors and the displacement of 25 m$^3$ for a well with a flow rate of 60 m$^3$/day. As a result, the profile of the exported concentration of inhibitors (Figure 2) and the time of removal of the inhibitors to a given final concentration (10 and 5 mg/l) were determined.
The results of modeling the time of removal of inhibitors to achieve concentrations of 10 and 5 mg/l are presented in Table 1.

Table 1. The simulation results of the removal of Descum-2 inhibitors, mark H-3611-A, Descum-4, mark S and HEDP

<table>
<thead>
<tr>
<th>Inhibitor</th>
<th>The time to reduce concentration, day</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Up to 10 mg/l</td>
</tr>
<tr>
<td>Descum-4 Mark S</td>
<td>152</td>
</tr>
<tr>
<td>Descum-2 mark H-3611-A</td>
<td>143</td>
</tr>
<tr>
<td>HEDP</td>
<td>125</td>
</tr>
</tbody>
</table>

Considering that the minimum operating concentration for HEDP is 10 mg/l, and for Descum - S5 mg/l, the protection time of the well with the given parameters is 152 and 305 days, and for Descum-2, H-3611-A 5 mg/l protection time of the well with the given parameters is 143 and 270 days, respectively. By optimizing the volumes of displacement and the volume of crushed scale inhibitors Descum-2, mark H-3611-A and Descum-4, mark S, the difference in removal time can be reduced or corrected according to the results of field tests.

2. Results and Discussion

It was established that 10% solution with Descum-2 reagent H-3611-A and Descum-4 reagent S in the amount of 0.5 m³ per 1 m³ of daily production of water should be pumped for the organization of scaling protection with Descum-2 reagent of mark H-3611-A and Descum-4 reagent of mark S, using the injection method within 365 days with the expected reagent removal of at least 10 g/m³. After selecting a well and calculating the number of inhibitors, 50 m³ of bottom-water is pumped into the formation.

In this article, the input control and laboratory tests of Descum-2 inhibitors of mark H-3611-A and Descum-4 of mark S, produced by LLC Mirriko, were carried out, which showed that:

- Samples of the reagent were input control and correspond to the indicators stated in the specifications and passport for the party [15].
- The properties of Descum-2 inhibitors H-3611-A and Descum-4 S-mark correspond to the set of requirements of the regulatory documents of Rosneft for admission to pilot tests.
- The reagent is provided with a full package of permits.
Based on the aggregate results of the analysis, the reagent can be recommended for pilot testing for the protection of borehole equipment from scaling by the method of injection into the reservoir, as well as to simulate the injection of inhibitor solutions into the reservoir (squeeze treatment), time of well protection and calculation of the required back pressure and displacement (design technology).

Recommendations on the use of inhibitors are given.

To calculate the total amount (in m$^3$) of Descum-2 reagents, mark H-3611-A and Descum-4, mark S, for conducting pilot tests for 365 days using the injection method with the expected reagent removal of at least 10 g/m$^3$, volume of daily water flow rate of the selected well should be multiplied by 65 (50 l – the volume of 100% Descum-2 reagent of mark H-3611-A and Descum-4 of mark S + 30% loss in the reservoir).

3. Conclusion

Thus, this study of various scale inhibitors confirms their useful properties, the use of which prevents premature wear, increases the overhaul period, as well as the duration of uninterrupted operation of downhole equipment. The results suggest that this study is relevant and appropriate and has a practical application of the studied ISO in the West Agan field of Western Siberia.

4. Conflicts of Interest

The authors declare no conflict of interest.

5. References


Effect of Mould Size on Compressive Strength of Green Concrete Cubes

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Received 02 February 2019; Accepted 29 April 2019

Abstract

This paper is aimed to evaluate the effect of mould size on compressive strength of concrete cubes made with recyclable concrete aggregates. Natural coarse aggregates were replaced with 50% recycled aggregates from old demolished concrete. Five different mould sizes were used to cast 420 concrete cubes using 1:2:4 mix and 0.55 water/cement ratio. In each size equal number of cubes was cured for 3, 7, 14 and 28-day. After curing, weight of cubes was determined followed by testing for compressive strength in universal load testing machine with gradually increasing load. From the obtained results the strength correction coefficients were computed keeping 28-day cured standard size cubes as control specimens. Also, numerical expression based on regression analysis was developed to predict the compressive strength using weight of cube, area of mould and curing age as input parameter. The numerical equation predicts the compressive strength very well with maximum of 10.86% error with respect to experimental results.

Keywords: Cube Size; Compressive Strength; Green Concrete; Old Concrete; Coarse Aggregate.

1. Introduction

Pace of construction of high-rise buildings around the globe is increasing day by day. It is mainly due to lack of space particularly around the city centers where need of space for accommodation and associated facilities is more in comparison to other parts of the country. To meet the need of construction of high-rise buildings in place of old and short height structures is unavoidable. Construction of new high-rise buildings not only consumes large quantities of natural aggregates but on other hand demolishing of old or short height structures generates huge quantum of demolishing waste. Proper disposal of this waste is another serious issue around the globe due to less available space for dumping.

One way of dealing this issue is by reusing it in new construction. This aspect remained active area of research among the researchers since couple of decades. Various components of demolishing waste have been successfully used in new construction. Coarse aggregates are the major component of concrete and occupy more space than other ingredients. Replacement of natural coarse aggregates in to or in percentage not only preserves the natural coarse aggregates but also reduces the waste disposal problem to some extent. Therefore, the use of old concrete as coarse aggregate in new concrete has got the insight of the researchers. Memon (2016) in his research article presented recent developments on
use of old demolished concrete as course aggregates in new concrete [1-3]. Quality assurance of new construction is ensured by material testing. Quality of concrete is mainly ensured by testing its compressive strength. To ensure this, specimens from running batch of concrete were prepared and cured for required time. ASTM defines procedure for making and curing concrete test specimens [4]. But different countries use different test specimen cube, cylinder or prism for compressive strength evaluation. Also depending on the loading capacity of testing machine available, different sizes of same specimens are in practice. Variation in strength results due to different shapes and sizes of specimens are evident from literature. Graybeal and Davis in their experimental work argued that strength testing of ultra-high-performance concrete is difficult due to machine capacity and cylinder end preparation [5]. Therefore, the authors used three different sizes of cylinders (2", 3" and 4") and cubes (2", 2.78" and 4") as an alternative specimen to test ultra-high-performance concrete in the strength range of 80-200 MPa. The authors used 51 samples of each cylinder and cube. From the experimental finding, authors observed that 3"cylinder and 2.78" and 4" cubes were acceptable as an alternative to standard cylinder specimen.

Although self-compacting concrete can be compacted under its own weight, yet any modification in mix design may affect the mechanical properties of the concrete. To this end Aslani in his research work studied the effect of mould size and shape on the compressive and tensile strength of self-compacting concrete with and without fibers. The authors in his experimental work used 100 mm and 150 mm cubes and 100×200 mm and 150×300 mm cylinders to test the compressive and tensile strength of plain SCC, steel- polypropylene and hybrid-fiber reinforced SCC concrete after 3-, 7-, 14-, 28- and 56-day curing. Based on the obtained results the author observed that effect of size on cube is more than cylinder. The author also developed numerical relationship based on fracture mechanics for size effect for cube and for cylinder [6]. Muhammad and Zuhair also conducted research work to check the effect of specimen size on strength of normal, high strength and self-compacting concretes. The authors conducted research in two phases. In first phase they cast cube and cylinder specimens of various sizes using locally available material. In second phase they analyzed the results and observed that variation of shape and size of specimen affects the strength of concrete. The authors concluded that correction factor for 150 mm to 100 mm cube for normal strength, high strength and self-compacting concrete are in the range of 0.89-1.29, 0.98-1.26 and 0.98-1.22 respectively. Similar values for standard cylinder to 100 × 200 mm cylinder were reported in the range of 0.88-1.08, 0.93-1.07 and 0.95-1.04 for normal, high strength and self-compacting concretes respectively [17]. Hemraj and Vikram conducted research work to explore the factors influencing the relationship between the cube and cylinder. The authors used standard sizes of cubes and cylinders to prepare the concrete specimens and cured them at 7 and 28 days. Based on the finding the authors concluded that there is no unique relationship between strength of cube and strength of cylinder [7].

Misba studied effect of size of cube mould on the compressive strength of concrete. Author used 4" and 6" cubes to cast the specimens. Based on compression tests results of specimen cured at 7, 14 and 28 days the author concluded that the 4" cube specimen gave more strength than 6" size specimen [8]. Matulic et al. for their research paper conducted experimental analysis of effect of size of test specimen on mechanical properties of shotcrete. The authors studied compressive strength and dynamic modulus of elasticity. The authors observed that the small size specimen results in higher compressive strength than large size specimen. Using the test results of dynamic modulus of elasticity, the authors also developed numerical expression to determine compressive strength of shotcrete [9].

In another study to check the effect of mould size and shape Alaa presented his research work using 260 specimens of different sizes from 30 high strength concrete mixes. Based on the experimental findings the author concluded that ratio of compressive strength of 150×300 mm cylinder to 150 mm cube is 0.8. For 100×200 mm cylinder to 150 mm cube the compressive strength ratio is 0.93. Finally, the author also pointed out that the compressive strength ratio of 150×300 mm cylinder to 100×200 mm cylinder is 0.86 [10]. Issa et al. in their research conducted to evaluate the size effect of cylinders on compressive strength of concrete, used 150×300 mm, 100×200 mm, 75×150 mm, and 50×100 mm size plastic moulds to cast 600 cylinders [11]. Experimental evaluation of compressive strength and modulus of elasticity revealed that co-efficient of variation of compressive strength increased as size of cylinder decreased.

Mansur and Islam conducted research to evaluate concrete strength for non-standard specimens. In their work the authors used five different sized of specimen and eleven batches of concrete to cast 210 cylinders and cubes. All the specimens were cured for same age and condition followed by testing them for the compressive strength. From the obtained results and their comparison with similar results available in literature the authors observed that strength ratio using standard cube to other specimen decreased with increase in concrete strength. Also decrease in strength was observed with decrease in size or aspect ratio of the specimen. The authors also developed numerical relationship for concrete strength, specimen size and specimen aspect ratio [12]. In another similar study by Ejiogu et al., for evaluation of effect of dimensions of sample on compressive strength of concrete samples, the authors used 170 concrete samples. They used 100, 150 and 200 mm size cubes, 100×150 mm cylinders and prism [13]. Based on the test results the authors concluded similar conclusion as given by Mansur and Islam [12]. Niloufaralso conducted research to check effect of size and shape of specimen on concrete strength at early and late age by PUNDIT (Non-destructive testing technique). Based on test results the author concluded that not only the shape but size of specimen also affects the strength of concrete. The author further observed that testing with non-standard size of specimen but at late age can diverge the
results [14]. Therefore, should be treated accordingly. Yi et al. conducted research to check the effect of mould size, shape and placement direction on compressive strength of concrete based on fracture mechanics. The authors used cube, cylinder and prism to test compressive strength of concrete. Observation from the test results shows that the effect of shape, size and placement direction is presented on compressive strength of concrete [15]. In another study by Krishna et al. conducted to study the effect of size and shape of specimen on compressive strength of glass fiber reinforced concrete, authors used standard cylinder, standard cube and 100×200 mm cylinder for the purpose [16]. Based on the obtained results the authors concluded almost similar conclusion as by Yi et al. [15].

Hasan et al. in their research work performed finite element modeling to check the effect of cube size on compressive strength of green concrete cubes is presented. The cubes were made with 50% replacement of natural coarse aggregates with coarse aggregates from demolished old concrete. The selection of 50% replacement was made based on the recommendations of Oad and Memon [2]. The details of material and testing is given in relevant section. From the obtained results correction coefficients due to mould size other than standard size (6”×6”×6”) were developed. These coefficients can be used to correct the strength equivalent to standard size cube. Also, numerical equation to predict the strength of cube based on mould area and weight of cube was developed using regression analysis. The predicted strength results using developed equation were in good agreement with experimental results.

2. Materials and Testing

Large blocks of demolished old concrete (Figure 1) were collected from balcony slab of a reinforced concrete school building about 50-year old situated in Nawabshah city. These blocks were manually hammered down to maximum size of 20 mm followed by screening of the material for cracked and damaged particles. After discarding cracked and damaged particles sieve analysis of the material was carried out following the standard procedure of sieving with maximum sieve size of 20 mm. Natural coarse aggregates (crushed) were also sieved using the same sieve sizes as used for recycled concrete aggregates. The result of sieving is plotted and compared in Figure 2.

Along with recyclable concrete aggregates from old concrete, crushed natural coarse aggregates, hill sand and ordinary Portland cement were used in preparation of concrete cubes using 1:2:4 mix and 0.55 water cement ratio. The concrete ingredients were batched using weight batching process. Five different sizes 2”×2”×2”, 3”×3”×3”, 4”×4”×4”, 6”×6”×6” and 9”×9”×9” of cube moulds were used to cast the cubes. As explained earlier both natural coarse aggregates and recyclable concrete aggregates from old concrete were used in 50% proportion. For each size of mould 84 cubes were cast, thus total of 420 concrete cubes were used in this research work. In each size of the mould equal number of cubes (21) was cured for 3, 7, 14 and 28 days by immersing in water.

After curing, cubes were allowed to air dry in laboratory for 24-hour, followed by determination of weight of each cube. For accurate determination of the results digital weight balance was used for the purpose. Finally, all the cubes were tested in universal load testing machine for compressive strength. The universal load testing machine is digital with display of applied load. The load was applied at the constant rate of 0.25 MPa per second till failure of the specimen. At failure the machine displays the peak load. The recorded failure load was then divided by the area of specimen to obtain the compressive strength of the specimen. The average values of weight and compressive strength of the cubes are given in Tables 1 and 2.
3. Results and Discussion

The compressive strength results obtained by using failure load and area of specimen in numerical expression of stress were analyzed. It is observed that the percentage difference between individual readings of stress within the group of specimens is less than 15% which shows compatibility of the results and proper casting of the specimens. Similar observation was also made for the weight of the cubes, the percentage difference between the weight of specimens remained within acceptable limits. Compressive strength vs cube numbers are plotted in Figure 3 for all curing ages considered in this research work. Sub plot (a) of this figure shows compressive strength trend all cubes for 3-day curing. Similarly, subplot (b) to subplot (d) shows the same for 7-, 14- and 28-day curing. From this figure it may be observed that the trend of compressive strength gain is similar in all cases.
(a) 3-day curing

Compressive Strength (3-Day cured cubes)

(b) 7-day curing

Compressive Strength (7-Day cured cubes)

(c) 14-day curing

Compressive Strength (14-Day cured cubes)
Figure 3. Compressive strength of all cubes

Figure 4 shows the average values of compressive strength at all curing ages for purpose of visualization and comparison. It may be observed that increase in compressive strength is recorded with increase in specimen size. Therefore, if any project site uses larger cube size in preparation of samples for strength testing will definitely result in higher strength values. These values may mislead the observer if size of the specimen is not declared. Therefore, strength correction should be applied. As the standard size of cube specimen is 6” x 6” x 6”, and standard curing is 28 days, therefore compressive strength relevant to above two parameters was taken as standard. Accordingly, the correction coefficients due to difference of mould size were evaluated and are given in Table 3. From this table it may be observed that as size of specimen reduces the correction coefficient becomes smaller and with increase in size of specimen than standard size coefficient becomes larger showing that the strength reported is more compared to the strength of standard size specimen. These coefficients may be used to equate the compressive strength of non-standard size cube to compressive strength of standard size cube cured at 28 days. The strength of non-standard size cube is divided with the coefficient given in table to obtain the strength equivalent to standard size cube.

In addition to above regression analysis was made to predict numerically the compressive strength based on weight of cube, curing age and area of mould. The predicted equation is given as under

\[
y = 8.9393 + 0.000479 \times A + 0.3467 \times C - 0.5102 \times W
\]  

(1)

In above equation dependent variable \( y \) represents target strength, \( A \) is area of mould, \( C \) denotes curing age (3, 7, 14 or 28) in days and \( W \) represents weight of the cube. In regression analysis significant \( F \) is the check variable which shows the reliability of the results and should be less than 0.5. In the current regression analysis both \( F \) and \( P \) values
obtained are less than 0.5. Also, the R-square value is equal to 96.9%. This shows the obtained equation is reliable in predicting the strength values. Using this equation compressive strength of all cubes tested for this research work was computed. It was observed that the equation predicted the compressive strength very well with minimum and maximum error equal to -10.72% and 10.86% respectively. Figure 5 shows the comparison of experimental observation of 4”×4”×4” cubes cured for 28 days vs the strength results obtained from numerical equation given in (1). It may be observed that the numerical equation produces strength results with similar trend to the experimental observation. However, for the particular comparison the predicted strength is about 4% less than experimental results.

During the testing process, the specimens were carefully observed for cracking and failure. Under gradually increasing load the crack started from top layer of the cube and propagated towards center and then towards bottom layer. With further loading first corners of the cube at top layer started reupting followed by rupture of the sides of cubes. At this point front and back faces of the cube were all cracked. Finally, the cube ruptured completely. Although old concrete as coarse aggregates is used in 50% dosage for preparation of the cubes, yet the failure mechanism of the cubes follows same pattern as of all-natural aggregate cubes. This shows viability of the old concrete as coarse aggregates in new concrete.

Table 3. Correction coefficients

<table>
<thead>
<tr>
<th>#</th>
<th>Size</th>
<th>Curing Age (Days)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>1</td>
<td>2”×2”×2”</td>
<td>0.42</td>
</tr>
<tr>
<td>2</td>
<td>3”×3”×3”</td>
<td>0.51</td>
</tr>
<tr>
<td>3</td>
<td>4”×4”×4”</td>
<td>0.56</td>
</tr>
<tr>
<td>4</td>
<td>6”×6”×6”</td>
<td>0.61</td>
</tr>
<tr>
<td>5</td>
<td>9”×9”×9”</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Figure 5. Comparison of experimental and numerical strength results of 4” side cube (28-day curing)

4. Conclusion

For this research work total of 420 concrete cubes with 50% recyclable old demolished concrete as coarse aggregate were cast using five different mould sizes to check the impact of mould size on compressive strength. After desired curing time, weight and compressive strength of all beams were determined. From the obtained results correction coefficients for compressive strength were determined by taking 150mm each size cube cured at 28-day as standard. These coefficients may be used to find compressive strength of non-standard size cubes equivalent to standard size cube cured at 28-day. Also, regression analysis was done to develop numerical equation to predict the compressive strength. Weight, area of cube and curing age were used as input parameters. The developed equation predicts the compressive strength in good agreement with experimental observations.

5. Conflicts of Interest

The authors declare no conflict of interest.
6. References


Behavioral Differences Towards Internal and External Factors in Making the Bid/No Bid Decision

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Received 26 February 2019; Accepted 16 May 2019

Abstract

Decision-making and subjective evaluation are two important aspects that characterizes the involvement of an organization in tender process. Moreover, selection of the right project to bid for is a principal feature of business success. The study aims to investigate the behavioral differences of Saudi construction contractors toward internal and external factors based on the process of modelling the bidding decisions. A quantitative research design is used to investigate the behavioral differences of 97 contractors recruited from construction industry of Saudi Arabia. A questionnaire was distributed among the respondents that would help in identifying the significant level of factors affecting the bid or no bid decision. The impact of internal and external factors on the bidding decisions was evaluated using one-way ANOVA analysis. The results have shown a significant and positive effect of internal and external factors on the bid or no bid decision; including job start time, work capital requirement, availability of qualified human resources, bidding methods, bidding document price, project supervision procedure and etc. The study has helped in establishing a better understanding toward the behavioral differences of contractors with respect to the bidding decisions.

Keywords: Behavioral Differences; Bid/No Bid Decision; External Factors; Internal Factors; Construction Industry.

1. Introduction

The customer tender enquiries need to be responded and handled carefully as it significantly affects the credibility and reliability of the organization. Majority of the organizations favor effective tender enquiry management. It is because of receiving customer tender enquiries that are directly proportional to bidding time. The company tends to bid more enquiries in time, when the company receives more customer tender enquiries [1]. Related to the factors affecting the company decisions, the decision of bid or no bid needs an understanding of company’s assessment. Every company has different assessment values. Therefore, the study aims to focus on the behavioral differences of Saudi Construction Contractors towards internal (qualified employees, plants and equipment, qualified subcontractors, and material suppliers) and external factors (contract, company experience, project characteristics, and characteristics of client) that affect the bid or no bid decisions.

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http://dx.doi.org/10.28991/cej-2019-03091323

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The overall engagement of an organization in tender process and decision to bid or no bid depends on the decision-making and subjective evaluation. It is believed that pre-bid screening and analysis procedure is a strategic tool for avoiding the circumstances, where many resources are involved. In such situations, decision-making solely depends on gut feeling [2]. An effective bid or no bid decision is likely to be valuable for a company through cost reduction and improved profits and revenues. The factors that affect the bid or no bid decision tend to differ in the level of importance that is assigned to each factor for taking a specific decision [3]. A study conducted by Lowe and Parvar [4] investigated the association between factors that affect bid and no bid decision and the decision taken by the organization. The results showed that out of 21, only 8 factors possessed significant linear association with the decision to bid. Wanous et al. [5] introduced artificial neural network, which was based on the input data that has been obtained from the contractors. The data mainly concerns about the factors that affect the bid or no bid decision, relates it with the output data, and formulates a final decision.

The existence of a contractor’s classification certificate is one of the main features of Saudi Arabia construction industry. The contractor classification system is responsible for helping contractors to bid for projects that are appropriate to them. The principal feature of business success is the selection of right project to bid for. In this context, the bid decision is important as it allow the contractors for evaluating their proposed projects to bid before acceptance getting committed to the bid-preparation processes. The modelling procedures depend on contractors’ assessment of each factor. The model user has not been identified by the previous studies, instead those studies developed a general model for all types of contractors. The bid/no bid decisions require an understanding of the human decision-making process and require processes of reaching to this decision in logical, systematic and organized manner. At the same time, supporting the decision-maker with the relative information needed to make the decision is required. Providing contractors with such a process and methodology seems to be more important than helping them to predict the decision without any rational basis. Therefore, the decision will be based on a logical perspective that would support the contractors in accepting it. The bid/no bid decision, as it is a repetitive decision, can be routinized to effectively use company’s experiences from past similar decisions. Then the likelihood of making “good” decisions can be increased.

The rationales behind selecting this topic are the researcher’s personal interest in generating knowledge about how contractors make decisions on issues related to their survival, how contractors practice the activities of strategic project management, and how the sharing culture can be applied within the managerial activities of a construction company. Investigating current procedure of bid/no bid decisions provided the researcher with the opportunity to gain this knowledge, as the decision proved to be one of the most critical decisions that contribute to the survival of the contractor. It is a strategic decision; it is a decision that requires a lot of information from different departments of the company, and it is an organizational decision that should be made with wide consultation. Therefore, the present study aims to investigate the behavioral differences of Saudi Construction Contractors toward internal and external factors on the process of modelling the bidding decisions. It investigates the behavioral differences based on different factors, since there is a paucity of studies accessible that examine the perceptions or attitudes of contractors toward factors of bidding process.

2. Literature Review

The contractors are continuously subjected to the external factors that influence their decision-making process. However, the association between international bidding and organizational culture in response to the economic and political risks of Malaysian international contractors is still a topic of research, regardless of the external factors. The models depicting actual bidding decision have been examined by a small number of qualitative studies. These studies mainly focus on the mark-up decision.

Oyeypio et al. [6] assessed the factors, affecting bidding decisions among contractors for construction projects. The study included 100 contractors to assess the factors, affecting bidding decisions. The findings showed insignificant impact of competition on the bidding decisions of contractors. The reputations of contractors are important in developing technical competencies and skills in assessing the competitiveness of contractors toward bidding decisions. Another study conducted by Wibowo et al. [7] analyzed the impact of bidding strategy on project and company performance. The study collected data from 61 contractors, using questionnaire survey and interviews and a structural equation modelling partial least squares (SEM-PLS) was used to analyze the data. The findings showed that project performance is directly affected through bidding strategies; while, company performance is indirectly affected through bidding strategies. The association between company performance and bidding strategy is positively mediated through project performance.

Patel et al. [8] studied the factors affecting the contractor’s bidding strategies in large scale residential construction. The data was collected through a questionnaire survey and individuals who were frequently involved in construction firms on daily basis were recruited. The sample included a total of 75 respondents that helped in identifying 49 factors under the group of contractors. Biruk et al. [9] focused on the set of tools for facilitating the main stages of competitive bidding process. The study used a linear programming model to assess contractor’s bidding decision. The findings have shown that the proposed approach stimulates the real-life bidding problems, based on accessible input and construction.
Enterprises. The in-house procedures and managerial decision support systems are ascertained through contract attractiveness and justifiable price.

Yan et al. [10] conducted a study to evaluate the individual, organizational, and group factors that are affecting the group bidding decisions for construction projects. The data was collected through a questionnaire survey among 203 Chinese international contractors. The results of the study identified 14 critical factors that affect the group bidding decisions for construction projects. Among the 14 critical factors, team decision preference and risk perception were found to be most significant. The study results were significant as it involved the understanding of various factors related to the organizational levels. These factors needed to be highlighted during the group bidding decision making process for better outcomes. The results narrated by Yan et al. suggested important strategies for bidding groups to enhance the decision-making process.

Petruseva et al. [11] predicted bidding prices in construction industry through support vector machine. The study enrolled 54 tenders from construction firms and the predictive modelling was analyzed through DTREG software. The findings have shown that internal and external factors affect the bidding process of a construction firm. Similarly, a critical and crucial decision is made by focusing on the bidding process. A similar study was conducted by Low et al. [12], who evaluated the impact of organizational culture on international bidding decisions. A culture-decision conceptual model was formulated and was tested with the help of a questionnaire survey that was further associated with interviews. The results suggested that the organizational culture was found to influence the international bidding decisions. However, this was not a prominent factor among risk decisions.

Shi et al. [13] developed a rough sets application and enhanced general regression neural network for assessing the uncertainty and influence of complex criteria. A MIBARK algorithm was adopted to assess the attribution reduction and decision table of rough sets. The findings have shown that generalization ability and prediction accuracy was enhanced through NPSO-GRNN. An effective bidding decision was made in uncertain construction markets through proposed decision support system. Enshassi et al. [14] identified and ranked the bidding decision based on contractor’s perspective. A total of 105 contractors were included in this study from Gaza Strip construction sector. The findings showed that the most critical factors affecting the bidding decision included: financial values, due data of the payments, stability of the construction industry, the financial capability of the clients, the financial capability of the contractors and the accessibility of construction raw materials. These findings tend to support the accessibility of bidding decisions in the construction industry.

Chen et al. [15] explored the relationship among risk perception, bidding decision-making, and risk propensity in the construction projects. A total of 134 contractors were included in the study. The data was collected using survey questionnaire and multivariate statistical analysis was used to analyze the collected data. The results showed that bidding decisions are negatively influenced by risk perception; while, it is positively influenced by risk propensity. Olatunji et al. [16] investigated the factors affecting the indigenous construction contractor’s decision on the bidding decisions. A total of 64 engineering management employees are included from Nigerian construction industry. The findings revealed the critical factors that affect bidding decision including; subcontracted work, difficulty in obtaining finance, business capacity of partners, resource price fluctuation, and own work. The aforementioned studies have indicated the impact of internal and external factors on the bidding decisions of construction industry. These findings have ascertained a platform to assess the behavioral differences of contractors toward internal and external factors in the construction industry. The hypothesis postulated in the present study are as follows;

Ho: There is no behavioral difference between Saudi contractors depending on their characteristics.
H1: There is a behavioral difference between Saudi contractors depending on their characteristics.

3. Methodology

The customer tender enquiries need to be responded and handled carefully as it significantly affects the credibility and reliability of the organization. Majority of the organizations favor effective tender enquiry management. It is because of receiving customer tender enquiries that are directly proportional to bidding time. The company tends to bid more enquiries in time, when the company receives more customer tender enquiries (Oduoza and Xiong, 2009). Related to the factors affecting the company decisions, the decision of bid or no bid needs an understanding of company’s assessment. Every company has different assessment values. Therefore, the study aims to focus on the behavioral differences of Saudi Construction Contractors towards internal (qualified employees, plants and equipment, qualified subcontractors, and material suppliers) and external factors (contract, company experience, project characteristics, and characteristics of client) that affect the bid or no bid decisions.

3.1. Research Design

A quantitative research design is used to investigate the behavioral differences of Saudi construction contractors toward the internal and external factors, affecting the bidding decision process in the Saudi construction industry. A
purposive sampling technique was used to enroll and collect data from contractors. Quantitative research design has been adopted in this study as it is an effective method for the under controlled conditions. Moreover, the research design is substantial for the studies which involve numbers. The data obtained from this research design aids in formulating importance information relating to the business decisions.

3.2. Data Collection

Survey approach has been incorporated in this study to collect data through a written questionnaire. The study targeted the construction and maintenance contractors in Saudi Arabia that include the managers and project managers. In the first step, the demographic details of all the respondents including firms’ status, names, addresses, degree of classification, and specialty were recorded. A total of 97 respondents participated and were asked about the company characteristics. The respondents were told to assess the effectiveness of internal and external factors in the judgment and experience related to bid and no bid decision. This helped in identifying the importance level of factors that affect the bid or no bid decision.

3.3. Instruments

A survey questionnaire has been used to quest the behaviors of Saudi contractors toward the internal and external factors. A total of 25 factors were identified from past studies, which were included in the survey questionnaire. These factors were rated on a six-point rating scale (0 indicates less influential; 6 indicates very influential).

3.4. Data Analysis

The data was analyzed through Statistical Package for Social Sciences (SPSS). Descriptive statistics was used for analyzing the characteristics and demographics variable of the respondents. Moreover, cross-tabulation was used to indicate the percentage of each factor and its impact on the bidding process. The data structure and causal modelling were discovered using principal component analysis.

4. Results

The internal and external factors are rated on a rating scale (0 – 6) showing the understanding of contractors and importance of these factors in construction projects. Table 1 has illustrated the rating of internal and external factors that reveals their importance in bidding decisions. The factors affecting bidding decision include; job start time, client requirements, cost of preparing the bid, original price, availability of required cash and equipment, bidding document price, and availability of labor.
ANOVA test has been performed for identifying the causal relation between internal and external factors in the bid/no bid decision-making process. The measurement of the significance of internal and external factors was computed through 0.05 level of significance. Table 2 has shown ANOVA findings for the internal and external factors associated with the bidding/no bidding process. The results showed significant impact of internal and external factors on bidding or no bidding decisions. In addition, the results have shown significant effects of job start time, location of the project, public exposure, client requirements, and work capital requirement, original price estimated by the client, degree of difficulties, availability of required cash, ability of doing the job, availability of required equipment, and availability of qualified human resources. Similarly, findings have shown significant effects of bidding document price, bidding methods, availability of labor, type of equipment required and the project supervision procedure on bidding process. Therefore, it has been implied that these internal and external factors play a vital role in making bid/no bid decisions in construction projects. The view that suggests that different contractors’ type have different weights of importance of the 87 factors is statistically supported. This suggests that at the stage of modelling the bid/no bid decision, the model user must be specified in terms of his/her characteristics. The most influential characteristics of the contractors that affected their assessment of the factors’ weights of importance were contractor size, classification status of the contractor and the main client type.
<table>
<thead>
<tr>
<th>S. No</th>
<th>Factors</th>
<th>Sig.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Job start time</td>
<td>0.032</td>
</tr>
<tr>
<td>2</td>
<td>Location of the project</td>
<td>0.034</td>
</tr>
<tr>
<td>3</td>
<td>The responsibility of issuing the work permits</td>
<td>0.011</td>
</tr>
<tr>
<td>4</td>
<td>Public exposure</td>
<td>0.008</td>
</tr>
<tr>
<td>5</td>
<td>The client requirements</td>
<td>0.008</td>
</tr>
<tr>
<td>6</td>
<td>Contract conditions</td>
<td>0.008</td>
</tr>
<tr>
<td>7</td>
<td>The cost of preparing the bid</td>
<td>0.010</td>
</tr>
<tr>
<td>8</td>
<td>Work capital required to start the job</td>
<td>0.000</td>
</tr>
<tr>
<td>9</td>
<td>Original price estimated by the client</td>
<td>0.029</td>
</tr>
<tr>
<td>10</td>
<td>Degree of difficulties in obtaining bank loan</td>
<td>0.009</td>
</tr>
<tr>
<td>11</td>
<td>Availability of required cash</td>
<td>0.002</td>
</tr>
<tr>
<td>12</td>
<td>Ability of doing the job</td>
<td>0.038</td>
</tr>
<tr>
<td>13</td>
<td>Availability of required equipment</td>
<td>0.037</td>
</tr>
<tr>
<td>14</td>
<td>Availability of qualified human resources</td>
<td>0.007</td>
</tr>
<tr>
<td>15</td>
<td>Need for work</td>
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<tr>
<td>16</td>
<td>General (office) overhead</td>
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</tr>
<tr>
<td>17</td>
<td>Reliability level of subcontractors</td>
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<tr>
<td>18</td>
<td>Required bond capacity</td>
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</tr>
<tr>
<td>19</td>
<td>Bidding document price</td>
<td>0.001</td>
</tr>
<tr>
<td>20</td>
<td>Bidding methods</td>
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</tr>
<tr>
<td>21</td>
<td>Availability of labour</td>
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<tr>
<td>22</td>
<td>Size of contract in SR</td>
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<tr>
<td>23</td>
<td>Location of the project</td>
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</tr>
<tr>
<td>24</td>
<td>Type of equipment required</td>
<td>0.018</td>
</tr>
<tr>
<td>25</td>
<td>The project supervision procedure</td>
<td>0.018</td>
</tr>
<tr>
<td>26</td>
<td>Safety hazards</td>
<td>0.002</td>
</tr>
</tbody>
</table>

5. Discussion

The findings have shown significant behavioral differences of Saudi Contractors toward internal and external factors associated with the bidding process of the construction industry. This result indicates that the classified contractors are more appreciative of the project selection phase than the non-classified contractors. That is not surprising as they have to consider projects that would help them in the renewal stage of their classification certificate. Therefore, it can be claimed that there is direct (positive) correlation on the level of importance given to the project selection stage and respondents’ organizational size. This would support the view that small and medium-size contractors are in need of a model or a decision aid to help them to become expert in making decisions on the processes involved in project selection. Different contractor types respond differently concerning the evaluation of projects. This result is surprising as the previous question (on importance of project selection) identified significant variance in the responses regarding the size of the contractor’s group. This result indicates that the small and medium size contractors have to ensure that the proposed project is within their capacity, so they evaluate the project before it is accepted. The non-classified contractors are more certain of their need of a decision aid. This result conflicts to some extent with their responses to this question. This may be due to whether they think they have adequate experience to make this decision, or due to a general resistance to change. The contractors involved in construction work are more certain in their responses than the contractors involved in maintenance work. That is because the bid/no bid decision for construction works could be more critical than the bid/no bid decision for maintenance works in term of project process and project development.

The findings of the current study are supported by several past studies. For instance, Oyeyipo et al. [6] have shown that the most important factors are accessibility of capital and material, and financial capability of clients. Thereby, the study has recommended that reputations of clients are developed through technical competencies and abilities in the construction industry. Similarly, the competitiveness of contractors is an important indicator in successful bidding process in construction projects. Another study Biruk, Jaśkowski & Czarnigowska [9] has shown the importance of behavioral differences in bidding process. The study indicated that the adoption of decision-support models and linear programming models are important for computing the bid amount and the total price among the items. These models
have been important in maximizing the cash flows of contractors in reference to the bill of quantities. Thereby, the study has recommended the use of decision-support models for modelling decisions of contractors in bidding processes.

Lim & Yazdanifard [17] have studied the influence of internal and external factors on the behavioral differences of contractors in bidding process. The results depicted that innovative ideas should be generated to expand profit for the marketers from online market. Holla et al. [18] have studied the influence of behavioral differences for 59 factors concerned with the analysis of bid selection. The study showed that the initial bid selection can eliminate the incompatible bid invitations by identifying the impact of behavioral differences among the contractors bidding selection process. This elimination can increase the valuable time for bid selection and preparation with improved chances of winning for bid proposals.

A study similar to the present study indicated that the bidding price directly influence the selection of construction firm business [11]. The study used a predictive modelling for predicting the bidding process of a construction project. The results showed that the selection of bidding process in the initial phase can be beneficial for contractors in increasing the outcomes of construction projects. However, Sandberg [19] depicted the importance of bidding process development in the case company. The study asserted that the benchmarking bidding process is sought to overcome the bidding process for best practices. The bidding process in the initial phase has been appropriate that enhance bid project management, bid competitions, and generate bids of higher quality to help the case company save resources.

Al-Alawi [20] indicated significant impact of behavioral differences of individual investors on the bidding process of construction industry. The study showed that economic and social factors were significantly affected on the decision-making process of investors in the Kingdom of Saudi Arabia. Moreover, the study has asserted that political factors and cultural factors are influential on the bidding process of investors; while, environmental factors and corporate governance influence the factors of investors negatively. On the basis of aforementioned discussion, it has been understood that internal and external factors contribute their part essentially on bidding decisions of stakeholders. These factors have emphasized a direct and significant impact of these factors on the bidding decisions. Thereby, it is important for construction industry to importantly measure the impact of these factors while bidding the projects.

6. Conclusion

The study has investigated the behavioral differences of Saudi construction contractors toward internal and external factors on the process of modelling the bidding decisions. These factors include job start time, location of the project, public exposure, client requirements, and work capital requirement, original price estimated by the client, degree of difficulties, availability of required cash, and ability of doing the job, availability of required equipment, and availability of qualified human resources. The study has established a better understanding toward the behavioral differences of contractors with respect to the bidding decisions. It is clear that internal and external factors impact behavioral differences of contractors during the bidding process. Thereby, it is important for construction industry of Saudi Arabia to consider these factors due to their direct relationship in the bidding decisions of contractors. Future studies should investigate these variables by undertaking diverse sample size or countries. Semi-structured interviews can be a vital tool for exploring perceptions and views of contractors regarding the association of internal and external factors during bidding decisions to gain in-depth understanding.

7. Conflict of Interest

The authors declare no conflict of interest.

8. References


Rapid Performance Evaluation of Water Supply Services for Strategic Planning

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Received 27 December 2018; Accepted 19 April 2019

Abstract

The assessment of existing water supply services was carried out through selected performance indicators with the aim of using that data in future for strategic planning of urban Mardan. The key performance indicators studied were selected to assess both the quantity and quality of water. The quality of water was assessed by turbidity, pH, and E.coli tests for samples collected at the start, middle, and tail end of the distribution system. The quantity of water supplied was measured by calculating discharges from water tapes at the three selected locations in the distribution system. A total of thirty samples were collected from ten union councils out of fourteen covering urban Mardan. A number of issues are highlighted in the overall water supply infrastructure and short, mid, and long term remedial measures are recommended. The results are presented in the form of an interactive map using Google Earth and VBA based dynamic database. It was found that the overall quality of water is generally acceptable for drinking. However, the presence of bacteria is an issue in many cases which needs to be resolved. A significant decrease in discharge is observed in the distribution systems away from the source due to leakages and illegal connections. A comprehensive overhaul of both management and infrastructure is required for sustainable and satisfactory level of services.

Keywords: Key Performance Indicators; Turbidity; pH; E.coli; Discharge; VBA.

1. Introduction

Domestic and municipal water needs are fulfilled by ground water schemes in most cities of Pakistan. Tube wells are one of the most common type of water extraction method adopted for municipal water supply schemes. Ground water is pumped to the surface and distributed to the communities through a distribution system using different types of piping systems. These water supply schemes and network of pipelines are decades old in many cases. New interventions have also been made under the impact of increasing population and expanded coverage areas. As such the water services providers rely both on the newly constructed water supply schemes and existing old schemes. These water supply systems in various cities of Pakistan face many problems related to maintaining proper water supply to all communities and regarding quality of water being delivered.

The cities of Pakistan are growing rapidly in size and population due to high rates of population growth and migration from the rural areas. In most cases, the growth is random and involves no planning. This causes various issues with...
regards to provision of public utilities. There is a need for condition assessment of the existing water supply infrastructure to propose necessary interventions for meeting the present and future demands of these ever growing urban centers.

Surface and groundwater sources for drinking water are contaminated by coliforms, toxic metals, and pesticides throughout Pakistan. Pakistan is ranked 80 among 122 nations regarding drinking water quality [1]. A study in District Bannu found significantly higher values of total dissolved solids (TDS) and electrical conductivity (EC) than World Health Organization (WHO) permissible limits [2]. A review of 43 studies (>9882 groundwater samples) revealed that 73% of the samples had arsenic values higher than WHO permissible limit (10 μg/L) [3]. In District Charsadda, the concentrations of nitrate exceeded the permissible limit (10 mg/L) set by US-EPA at 13 sites, while concentrations of sulfate exceeded the permissible limit (500 mg/L) of WHO at 9 sites [4].

The general perception about water supply services is negative in most cities of Pakistan. In district Vehari of Punjab province, 48.6% of people disagreed that the drinking water of their area is good. These people reported more disease development (45.8%) compared to those who were satisfied (11.1%) with their drinking water quality [5]. However, the northern mountainous regions generally have good quality of drinking water. A study of 44 samples in Bajaur agency showed that hydrochemical characteristics and quality assessment parameters were within the permissible range set by WHO except SO₄²⁻, K⁺, NO₃⁻, and HCO₃⁻ in 9%, 40%, 54%, and 67% of the analyzed samples, respectively [6]. A study conducted in Swat valley concluded that the physicochemical parameters were within their safe limits except in a few locations, whereas, the fecal contaminations in drinking water resources exceeded the drinking water quality standards of Pakistan Environmental Protection Agency (Pak-EPA), 2008 and World Health Organization (WHO), 2011 [7].

The water industry needs to develop values and a system wherein key performance indicators are used to identify areas of improvement, define rational targets, devise action plans and track improvements over time. One such system is to define and evaluate key performance indicators (KPIs) for a given water supply scheme [8]. KPIs are the fundamental components of a groundwork process whereby water utilities can assess their relative technical and financial performance against internationally defined standards [9]. They can be used by the water supply services providers for appropriate decision making and need based response. However, many of the internationally adopted systems involve so many indicators which make them too comprehensive and exhaustive to be utilized in developing countries such as Pakistan [10]. Therefore, there is a dire need for a more practical and economical system to meet the needs of water supply schemes in developing countries.

2. Study Area and Research Methodology

This research focuses on identifying easily obtainable key performance indicators for drinking water supply schemes in the urban settlements of Mardan city, present the data gathered in an easily accessible and comprehensible fashion, and to suggest measures for improving water supply services for consumers. Drinking water supply schemes in the fourteen union councils of Mardan city are studied. The field investigation part of the study focuses on developing a data collection tool in the form of a pro forma questionnaire, measurement of discharge values at different points in the distribution system, identification of issues in the infrastructure causing deterioration of the water supply services, and recommendations of remedial measures. The research will be helpful for improving the monitoring techniques in the existing system.

The step by step methodology adopted in this research study is shown in Figure 1. A pro forma containing questions related to condition assessment of the schemes was prepared and used in the field for data collection. Samples at three points in the distribution systems were collected for laboratory tests. Physical, chemical, and bacteriological tests were then performed and results analyzed.

Three laboratory tests, namely turbidity, pH, and total coliform are performed to assess the water quality of the selected schemes. Water samples at the start, mid, and tail end of each distribution system are collected and tested. The parameters selected represent a measure of both suspended solids and microorganisms in water. More parameters may be added if required and represented in a similar fashion.

In the last phase of the research an interactive map was developed based on Google Earth and using VBA based open source database application called Mapper. The user can select any parameter investigated in this study and the results are displayed for all the sampling points on a Google Earth environment. The database can also be easily extended to include more parameters in the future. This is an important development because it will help the non-technical decision makers understand engineering data easily.
3. Condition Assessment of Existing Infrastructure

A number of issues were observed in the overall water supply infrastructure and management during the field investigation part of this research study. Highlighting these issues is a first step towards a systemic improvement of the water supply services in urban centers of Pakistan. It is observed that the unplanned growth and trend of urbanization is also a major factor in adversely affecting the water supply infrastructure. Most of the city area is congested with narrow streets lacking proper water supply system, lack of appropriate sanitation arrangements and deficient cleaning provisions. Since the water supply is intermittent, many inhabitants have installed sucking machines to fill overhead tanks or ponds in their homes. Resultantly, the tail users are unable to receive the required quantum of water.

The managing authority of the water supply schemes have deployed a considerable number of employees for running and maintenance purposes. However, due to lack of proper tools, equipment, generators as an alternate power source and vehicles for logistics the services level could not be termed as satisfactory. The required skill was mostly available but due to lack of funds and absence of required facilities, the system could not run smoothly and to the desired satisfactory level. There is no appropriate strategy for line crossing through roads and irrigation channels depriving several areas and households from the service due to non-availability of crossing facilities. Sharp edges in pipes were observed which can cause more head losses as shown in Figure 1(a). Sometimes, the whole water supply scheme is nonfunctional as shown in Figure 1(b) due to lack of maintenance of critical equipment such as pressure pump as shown in Figure 1(c). In many cases, the distribution pipes and connections to consumers are passing through sewerage lines as shown in Figure 1(d). This can cause induction of bacteria and other impurities into the drinking water.
Since almost all the units are provided for a specific union council and there is no networking among different units except in few cases, the inhabitants are left without water supply in case the system fails due to one reason or the other. Similarly with the exception of few union councils, there are no alternate water supply arrangements. As a result, people try to get the required water through their own sources. One such arrangement is the extraction of ground water using private bore holes and pressure pumps. This puts an extra burden on the ground water resources in the area and many places are reporting lowering of ground water table.

4. Recommendations for Improvement

The service level can be greatly improved by taking necessary short-term, mid-term, and long-term measures. All the non-operational and damaged units should be repaired and rehabilitated to ensure adequate usage of the already installed infrastructure. In many cases, the rehabilitation cost is negligible as compared to the cost of developing a new scheme. Alternate power supply system i.e. generators, solar panels etc. needs to be installed to make sure a continuous power supply to the pumping units. The staff employed on site at these units are non-technical people and they have to call technicians for all kinds of minor and major issues which may develop and interrupt the supply of water. This process takes a long time in most cases. Therefore the onsite personnel should be trained so that they can take care of small repairs in the units and avoid unnecessary interruptions in water supply. All the illegal connections and sucking machines should be prohibited and strict action should be taken against those violating the rules in this regard.

Adequate machinery, vehicles, and equipment related to proper running of water supply services should be procured. Appropriate budgetary provision is needed in this regard in the government annual development program (ADP). The water losses (both real and apparent) are at unacceptably high levels, which threatens the water resources and also results in excessive energy costs for pumping and transmission. Therefore, reduction of real and apparent losses must become a key priority for the management.

A detailed master planning of Mardan city should be carried out to identify necessary measures for improving existing infrastructure and propose new schemes to meet the needs of future population. This should be made a regular exercise every five years. Proper networking among the schemes should be developed to ensure continuous water supply in case one of the units becomes non-operational. The water supply agencies operate financially in a completely unsustainable way as they are largely depending on the government subsidies with a very nominal share received from users in shape of water toll collection. Use of the flat rate system with un-measured amount of water does not ensure the full cost recovery of the services. To provide sustainable services, the concerned authorities must place greater focus on financial sustainability by maximizing revenue generation and reducing the costs of operation.

5. Results and Discussions

The discharge data for all three collecting points of each tube well is shown in Table 1. As can be seen, the discharge is not uniform for all the tube wells at a corresponding point in the distribution system. The maximum discharge was recorded at the start point of Bijligar union council with a value of 2769.2 G/day. The minimum discharge was recorded at the tail point of Nala Par Hoti union council with a value of 138.5 G/day.

The average discharge at start, middle, and tail points were calculated to be 1498.9 G/Day, 737.9 G/Day, and 487.8 G/Day respectively. The last column of Table 1 shows the percent decrease in discharge which occurs from start point to the tail point. A significant decrease in discharge is observed in all the tube wells from start point to the end point. The decrease is over 50 percent in all cases except one. The maximum drop in discharge was observed to be 80 percent in the Bijligar tube well. The average decrease was found to be 63.60 percent.
Table 1. Discharge data for each tube well

<table>
<thead>
<tr>
<th>S. No</th>
<th>Union Council</th>
<th>Discharge G/day</th>
<th>Percent Decrease from Start to Tail</th>
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<tr>
<td></td>
<td></td>
<td>Start Point</td>
<td>Middle Point</td>
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<tr>
<td>1</td>
<td>Muslim Abad</td>
<td>744.8</td>
<td>653.4</td>
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<td>2</td>
<td>Mardan Khaas</td>
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<td>800.0</td>
</tr>
<tr>
<td>3</td>
<td>Nala Par Hoti</td>
<td>469.6</td>
<td>223.0</td>
</tr>
<tr>
<td>4</td>
<td>Barichum</td>
<td>1542.8</td>
<td>813.6</td>
</tr>
<tr>
<td>5</td>
<td>Jan Abad</td>
<td>1350.0</td>
<td>919.8</td>
</tr>
<tr>
<td>6</td>
<td>Bicket Gung</td>
<td>2160.0</td>
<td>986.0</td>
</tr>
<tr>
<td>7</td>
<td>Bughdada</td>
<td>2160.0</td>
<td>986.0</td>
</tr>
<tr>
<td>8</td>
<td>Dhagai</td>
<td>900.0</td>
<td>390.0</td>
</tr>
<tr>
<td>9</td>
<td>Bijligar</td>
<td>2769.2</td>
<td>872.0</td>
</tr>
<tr>
<td>10</td>
<td>Kas Korona</td>
<td>1542.8</td>
<td>735.8</td>
</tr>
</tbody>
</table>

Figure 2 shows the variation in discharge recorded for each tube well. There is a consistent drop in discharge for all tube wells from start point to end point. Leakages and illegal connections are the main reasons for the decrease in discharge.

![Discharge data for each tube well at three points](image)

Table 2. Results of Turbidity tests

<table>
<thead>
<tr>
<th>S. No</th>
<th>Union Council</th>
<th>Turbidity (NTU)</th>
<th>Comparison with WHO Standard (&lt; 5 NTU)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Start Point</td>
<td>Middle Point</td>
</tr>
<tr>
<td>1</td>
<td>Muslim Abad</td>
<td>3.2</td>
<td>2.1</td>
</tr>
<tr>
<td>2</td>
<td>Mardan Khaas</td>
<td>3.2</td>
<td>3.4</td>
</tr>
<tr>
<td>3</td>
<td>Nala Par Hoti</td>
<td>2.6</td>
<td>2.4</td>
</tr>
<tr>
<td>4</td>
<td>Barichum</td>
<td>2.6</td>
<td>3.2</td>
</tr>
<tr>
<td>5</td>
<td>Jan Abad</td>
<td>2.4</td>
<td>3.0</td>
</tr>
<tr>
<td>6</td>
<td>Bicket Gung</td>
<td>2.0</td>
<td>2.1</td>
</tr>
<tr>
<td>7</td>
<td>Bughdada</td>
<td>2.7</td>
<td>2.7</td>
</tr>
<tr>
<td>8</td>
<td>Dhagai</td>
<td>2.4</td>
<td>2.2</td>
</tr>
<tr>
<td>9</td>
<td>Bijligar</td>
<td>1.4</td>
<td>2.2</td>
</tr>
<tr>
<td>10</td>
<td>Kas Korona</td>
<td>2.2</td>
<td>4.3</td>
</tr>
</tbody>
</table>
The bar charts in Figure 3 show turbidity results for three collection points of each tube well. The WHO limit is crossed only at the end point of Jan Abad union council as shown in the figure.

The procedure outlined in ASTM D1293-12 [12] was followed during the pH test. The results obtained are shown in Table 3. Only one sample was found to have pH value outside the recommended range of WHO standards. The sample taken at the starting point of Bicket Gung scheme has a pH value of 9.4 which is greater than the acceptable upper limit of 8.5.

### Table 3. pH test results

<table>
<thead>
<tr>
<th>S. No</th>
<th>Union Council</th>
<th>pH Start Point</th>
<th>pH Middle Point</th>
<th>pH Tail Point</th>
<th>Comparison with WHO Standard (6.5-8.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Muslim Abad</td>
<td>7.1</td>
<td>7.0</td>
<td>7.4</td>
<td>Acceptable</td>
</tr>
<tr>
<td>2</td>
<td>Mardan Khaas</td>
<td>7.5</td>
<td>7.5</td>
<td>8.0</td>
<td>Acceptable</td>
</tr>
<tr>
<td>3</td>
<td>Nala Par Hoti</td>
<td>5.5</td>
<td>6.3</td>
<td>8.0</td>
<td>Not Acceptable</td>
</tr>
<tr>
<td>4</td>
<td>Barichum</td>
<td>7.4</td>
<td>7.5</td>
<td>7.8</td>
<td>Acceptable</td>
</tr>
<tr>
<td>5</td>
<td>Jan Abad</td>
<td>8.0</td>
<td>6.0</td>
<td>8.0</td>
<td>Not Acceptable</td>
</tr>
<tr>
<td>6</td>
<td>Bicket Gung</td>
<td>9.4</td>
<td>7.0</td>
<td>7.6</td>
<td>Not Acceptable</td>
</tr>
<tr>
<td>7</td>
<td>Bughdada</td>
<td>7.2</td>
<td>7.3</td>
<td>7.8</td>
<td>Acceptable</td>
</tr>
<tr>
<td>8</td>
<td>Dhagai</td>
<td>7.6</td>
<td>8.0</td>
<td>8.0</td>
<td>Acceptable</td>
</tr>
<tr>
<td>9</td>
<td>Bijligar</td>
<td>6.3</td>
<td>8.0</td>
<td>8.0</td>
<td>Not Acceptable</td>
</tr>
<tr>
<td>10</td>
<td>Kas Korona</td>
<td>7.5</td>
<td>6.0</td>
<td>7.9</td>
<td>Not Acceptable</td>
</tr>
</tbody>
</table>

The pH values along with WHO limits are represented by the bar charts in Figure 4. Most of the pH values fall within WHO recommended range. However, values recorded at some points are less than the min limit of WHO guidelines.

**Figure 4. Results of turbidity test for each union council**

**Figure 5. pH values at selected points of each union council**
Table 4 shows the results obtained for all samples from the bacteria test [13]. Water of two units is completely free from bacteria, three units have traces of bacteria all along the distribution system, and the remaining units have traces of bacteria at some point(s) in the distribution system. These results indicate that the source of water is generally free from microorganism and the bacterial contamination is resulted from external impurities. The obvious reasons are leaked pipes and exposed unhygienic passages of the distribution system. A domestic remedy for this issue would be to boil the water before drinking and usage in food preparation.

<table>
<thead>
<tr>
<th>S. No</th>
<th>Union Council</th>
<th>Total Coliform (cfu)</th>
<th>Comparison with WHO Standard (0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Start Point</td>
<td>Middle Point</td>
</tr>
<tr>
<td>1</td>
<td>Muslim Abad</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>Mardan Khaas</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>Nala Par Hoti</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>4</td>
<td>Baricham</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>Jan Abad</td>
<td>0</td>
<td>6</td>
</tr>
<tr>
<td>6</td>
<td>Bicket Gung</td>
<td>8</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>Bughdada</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>8</td>
<td>Dhagai</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>Biligar</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>10</td>
<td>Kas Korona</td>
<td>9</td>
<td>6</td>
</tr>
</tbody>
</table>

The water testing and monitoring should be a continuous process to ensure suitable water quality for drinking water. Any apathy towards water quality assurance may result in water borne diseases which is a serious issue in many developing countries including Pakistan.

6. Representation of Data

One of the major issues in the decision making process of public infrastructure development is the communication gap between engineering professionals and non-technical decision makers. Often times, the message is lost in the heavily technical engineering terminology and cause unnecessary delays in decision making or complete rejection. The data presented in the form of tables and graphs is hard to understand by the non-technical managers and government officials. Therefore there is always a need for tools and techniques to make the technical engineering data easily comprehensible to the common public. Development of an interactive map as part of this research was intended to serve this purpose.

The graphical representation of the data is achieved by Google Earth using the Mapper software application. Mapper is a dynamic VBA based open source database application. It was initially developed for mapping water resources by a computer programmer named Owen Scott. However, certain modifications pertaining to data collected for our research was carried out by changing certain codes in Excel developer. Anyone can use the application on a personal computer with Google Earth and Excel software. The input data can be easily modified and updated. The information displayed on the map can be easily modified by selecting from the list of given parameters. The water scheme is displayed as an icon on a Google Earth image and list of selected information shown. Figure 2 shows screen shot of a map created for functionality of schemes. Data for each scheme can displayed in a tabular form on the map by clicking on a particular scheme as shown in Figure 2.

![Screen shot of functionality map](image)

Figure 6. Screen shot of functionality map
7. Conclusion

The overall efficiency of the system is severely affected by the lack of proper maintenance arrangements as demonstrated by a number of issues found in the water supply infrastructure. About one-fourth of the schemes are in non-operational condition. Over 50 percent decrease in discharge was observed from start to end in the distribution system except for one scheme. The prime reasons were found to be leakages and illegal connections.

The turbidity level was found to be within the WHO acceptable limits for all the sampling points except at the tail point of Jan Abad distribution system. The pH values were found to be within the WHO acceptable limits of drinking water except for start point of Bicket Gung distribution system. Water of two units is completely free from bacteria, three units have traces of bacteria all along the distribution system, and the remaining units have traces of bacteria at some point(s) in the distribution system. The overall quality of water is generally acceptable for drinking. However, the presence of bacteria is an issue in most cases which needs to be resolved.

8. Acknowledgement

The first author is grateful for the financial support of Creative Engineering Consultants, Pakistan during this study.

9. Conflict of Interest

The authors declare no conflict of interest.

10. References


