Civil Engineering Journal (ISSN: 2476-3055)

Editor in Chief:
Prof. M. R. Kavianpour
K.N.Toosi University of Technology (Iran)

Executive Manager:
Dr. O. Aminoroayaie Yamini
K.N.Toosi University of Technology (Iran)

Dr. S. Hooman Mousavi
K.N.Toosi University of Technology (Iran)

Editorial Board Members:
Prof. Dintie S. Mahamah
St. Martin's University (USA)

Dr. Kartik Venkataraman
Tarleton State University (USA)

Dr. Tanya Igneva
University of ACEG (Bulgaria)

Dr. Daniele Bocchiol
Polytechnic University of Milan (Italy)

Dr. Michele Iervolino
Second University of Naples (Italy)

Dr. Rouzbeh Nazari
Rowan University (USA)

Prof. Marta Bottero
Polytechnic University of Turin (Italy)

Chris A. O’Riordan-Adjah (PhD Candidate)
University of Central Florida (USA)

Dr. Yasser Khodair
Bradley University (USA)

Dr. Weidong Wu
University of Tennessee - Chattanooga (USA)

Dr. Kaveh Saleh
University of Sherbrooke (Canada)

To view all editorial board members Click Hear.

Dr. Jiliang Li
Purdue University North Central (USA)

Dr. Yaqi Wanyan
Texas Southern University (USA)

Prof. M.M. Rashidi
Tongji University (China)

Dr. Sanjay Tewari
Louisiana Tech University (USA)

Prof. Nikolaos Eliou
University of Thessaly (Greece)

Dr. Mohammad Reza Najafi
University of Victoria (Canada)

Dr. Saeed Khorram
Eastern Mediterranean University (Cyprus)

Dr. Xinquen Zhu
University of Western Sydney (Australia)

Dr. Jalil Kianfar
St. Louis University (USA)

Dr. Luca Comegna
Second University of Naples (Italy)

Dr. Davide Dalmazzo
Polytechnic University of Turin (Italy)

Dr. Viviana Letelier González
University of the Frontera (Chile)

Dr. Paola Antonaci
Polytechnic University of Turin (Italy)

Dr. Davorin Penava
University of Osijek (Croatia)

Dr. Ali Behnood
Purdue University (USA)

Dr. Uğur Albayrak
Eskisehir Osmangazi University (Turkey)
Contents

- Page 2525-2534
  A Novel Buffer Tank to Attenuate the Peak Flow of Runoff
  Yinghong Qin, Zhengce Huang, Zebin Yu, Zhikui Liu, Lei Wang

- Page 2535-2553
  Moisture Susceptibility of Asphalt Concrete Pavement Modified by Nanoclay Additive
  Saif Al-din Majid Ismael, Mohammed Qadir Ismael

- Page 2554-2568
  Heavy Oil Residues: Application as a Low-Cost Filler in Polymeric Materials
  Yulia Yurevna Borisova, Alsu M. Minzagirova, Alina R. Gilmanova, Mansur F. Galikhanov, Dmitry N. Borisov, Makhmut R. Yakubov

- Page 2569-2578
  Behavior of Reinforced Concrete Beams with effect of Stiffened Plates
  Ali Sabah Al Amlin, Laith Shakir, Ali Abdulredha, Nodhir Al-Ansari

- Page 2579-2586
  A Quantitative Approach to Prioritize Sustainable Concrete
  L. Sudheer Reddy, A. Suchith Reddy, S. Sunil Pratap Reddy

- Page 2587-2597
  Structural Characteristics of Developed Sustainable Lime-Straw Composite
  Sajid Kamil Zemam, Sa’ad Fahad Resan, Musab Sabah Abed

- Page 2598-2613
  Investigating Role of Vegetation in Protection of Houses during Floods
  Amina Ali, Ghufran Ahmad Pasha, Usman Ghani, Afzal Ahmed, Fakhar Muhammad Abbas

- Page 2614-2625
  Parametric Study of the Modal Behavior of Concrete Gravity Dam by Using Finite Element Method
  Seyed Reza Jafari, Majid Pasbani Khiavi

- Page 2626-2642
  An Investigation of the Fundamental Period of Vibration for Moment Resisting Concrete Frames
  Ahmed Nader Mohamed, Khaled F. El Kashif, Hamed M. Salem

- Page 2643-2664
  An Evaluation of Barriers obstructing the Applicability of Public Private Partnership (PPP) in Infrastructure Development
  Sedqi Esmaeel Rezouki, Jinan Kata’a Hassan
Contents

- Page 2665-2685
  A New Approximate Method for Earthquake Behaviour of Worship Buildings
  Pınar Usta, Özgür Bozdag

- Page 2686-2699
  Photosynthetic Microbial Desalination Cell to Treat Oily Wastewater Using Microalgae Chlorella Vulgaris
  Suhad Shamil Jaroo, Ghufnan F. Jumaah, Talib R. Abbas

- Page 2700-2707
  Selection of the Optimal FBG Length for Use in Stress-Strain State Diagnostic Systems
  Viktor Vikulov, Aleksandr V. Todorov, Aleksey V. Faustov, Nikolay L. Lvov

- Page 2708-2726
  Finite Element Analysis of Beam – Column Joints Reinforced with GFRP Reinforcements
  Balamuralikrishnan R., Saravanan J.

- Page 2727-2737
  The Effect of Using Sustainable Materials on the Performance-Related Properties of Asphalt Concrete Mixture
  Amjad H. Albayati, Waleed Arrak Turkey

- Page 2738-2746
  Sensitivity of Direct Runoff to Curve Number Using the SCS-CN Method
  Abdul Ghani Soomro, Muhammad Munir Babar, Anila Hameem Memon, Arjumand Zehra Zaidi, Arshad Ashraf, Jewell Lund

- Page 2747-2761
  Structural Behavior of High Strength Laced Reinforced Concrete One Way Slab Exposed to Fire Flame
  Anas Ibrahim Abdullah

- Page 2762-2772
  Mineralogy, Micro-fabric and the Behavior of the Completely Decomposed Granite Soils
  Elsayed Elkamhawy, Bo Zhou, Huabin Wang
Focus and Scope

Civil Engineering Journal (C.E.J) is a multidisciplinary, an open-access, internationally double-blind peer-reviewed journal concerned with all aspects of civil engineering, which include but are not necessarily restricted to:

- Building Materials and Structures
- Constructions Technology
- Earthquake Engineering
- Renovation of Buildings
- Highway Engineering
- Road and Bridge Engineering
- Surveying and Geo-Spatial Engineering
- Tunnel Engineering
- Water Resources Engineering
- Coastal and Harbor Engineering
- Constructions Economy and Management
- Environmental Engineering
- Geotechnical Engineering
- Hydraulic and Hydraulic Structures
- Structural Engineering
- Transportation Engineering
- Urban Engineering and Economy
- Urban Drainage

Special Issues

Special Issues deal with more focused topics with high current interest falling within the scope of the journal in which they are published. Special Issue proposals are welcome at any time during the year.

For most of the civil engineering conferences it is possible to submit papers presented at the conference for subsequent publication in special issues of the C.E.J.

- Civil Engineering Journal (C.E.J) is published monthly.
- Civil Engineering Journal (C.E.J) has fast peer review process (3-4 weeks).

Civil Engineering Journal (C.E.J) Indexing & Abstracting

- This is an open access journal under the CC-BY license (https://creativecommons.org/licenses/by/4.0/).
A Novel Buffer Tank to Attenuate the Peak Flow of Runoff

Yinghong Qin a, b,*, Zhengce Huang c, Zebin Yu b, c, Zhikui Liu a, Lei Wang a

*College of Civil and Architecture Engineering, Guangxi University of Technology, 541004, Guilin, China.

College of Civil Engineering and Architecture, Guangxi University, 100 University Road, Nanning, Guangxi 530004, China.

Hualan Design & Consulting Group, Nanning, Guangxi Province, 530004, China.

Received 07 August 2019; Accepted 16 October 2019

Abstract

Impermeable pavements and roofs in urban areas convert most rainfall to runoff, which is commonly discharged to local sewers pipes and finally to the nearby streams and rivers. In case of heavy rain, the peak flow of runoff usually exceeds the carrying capacity of the local sewer pipes, leading to urban flooding. Traditional facilities, such as green roofs, permeable pavements, soakaways, rainwater tanks, rain barrels, and others reduce the runoff volume in case of a small rain but fail in case of a heavy rain. Here we propose a novel rainwater buffer tank to detain runoff from the nearby sealed surfaces in case of heavy rain and then to discharge rainwater from an orifice at the tank’s bottom. We found that considering a 100 m² rooftop with 0.80 runoff coefficient and a 10cm rainfall depth for an hour, a cubic tank with internal edge side of a square of 2 m attenuates the peak flow about 45%. To reduce a desirable peak flow, the outlet orifice of the buffer tank must be optimized according to site-specific conditions. The orifice can be set at an elevation from the tank’s bottom to create a dead storage for harvesting rainwater.

Keywords: Urban Flooding; Runoff; Rainwater Tank; Rainwater Management; Peak Flow.

1. Introduction

Urbanization has sealed natural permeable surfaces with pavements, roofs, and other impermeable surfaces. Rainwater falling on these surfaces generates runoff, which is diverted to local sewer pipes and finally ends up at nearby streams or rivers. In the case of a heavy rain, the runoff-discharging rate commonly exceeds the carrying capacity of the sewer pipes, resulting in urban flooding. Urban flooding subsequently causes a series of serious negative consequences such as traffic jams, loss of human life, damage to property, loss of livestock, and deterioration of health conditions owing to waterborne diseases, and others [1]. Across the globe, rainwater management techniques have widely employed to mitigate urban flooding. Mainstream techniques include Low Impact Development [2], Best Management Practices [3], Water Sensitive Urban Design [4], Sponge Cities [5], and other similar projects [6-8]. While their names of these projects are different, the purposes of these techniques are similar. That is, on-source techniques are developed to retain, detain, infiltrate, harvest, evaporate, transpire, and/or re-use rainwater for reducing the runoff volume and the peak flow.

The specific rainwater management techniques include green roofs, permeable pavements, bio-retentions, soakaways, rainwater tanks, rainwater barrels, and others. A green roof consists of a vegetated layer and a growing medium layer above a roof deck, over which both layers cannot further retain rainwater after they get saturated [9]. Green roofs therefore effectively reduce the runoff volume and the peak flow in case of a small rain but fail in case of a

*Corresponding author: yqin1@mtu.edu

http://dx.doi.org/10.28991/cej-2019-03091430

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
heavy rain [10, 11]. A permeable pavement consists of a permeable surface layer that allows rainwater passing through it to the base layer, where rainwater stores and infiltrates to the subgrade layer slowly [12]. Similar to green roofs, permeable pavements lead to overflow in case that the water-storing base saturates [13]. Soakaways are buried chambers filled with gravels, rubber, and stones that store water in the cavities of the fillers and discharge the water by infiltration [14]. The cavities of the fillers are limited so the buffering capacity of a soakaway is inadequate to store a large volume of water [15]. A bio-retention unit is a bowled land backfilled with a vegetated surface layer and with a filter intermediate layer upon natural soils. A bio-retention unit ponds rainwater in the bowled lands for subsequent infiltration [16]. As the infiltration is controlled by the permeability of the soils, overflow occurs in case of a heavy rain if the permeability is small [16, 17]. Rainwater tanks or rainwater barrels are small water-storage units collecting rainwater from nearby sealed surfaces [18]. While these rainwater storage units can store a great amount of water, they must be frequently emptied artificially for re-storing rainwater at the next rain [19, 20]. In seasons of heavy and intense rainfall, these units are commonly failed to be emptied [21]. Therefore, it is desirable if rainwater tanks or rainwater barrels can be emptied automatically before the next rainfall event.

Here we propose a novel water buffer tank to temporarily store rainwater in case of heavy rain. The design of this tank is schematically shown in Figure 1. For instance, the tank can be connected to a downspout to detain rainwater from a rooftop. The retained water in the tank can be discharged to nearby sewer pipes or gutters. The outlet orifice is set smaller than the inlet pipe. In case of small rain, this tank loses the buffering capacity because the inflow to the tank is equal to the outflow. In case of heavy rain, the outflow to the tank is smaller than the inflow, with some storing in the tank and the surplus discharging from the outlet orifice. The discharging rate increases as the water head in the tank increases. After rainfall stops, the detained water continually drains from the outlet orifice until the tank is emptied. Therefore, the tank serves as a buffer to attenuate the peak flow. By optimizing the outlet orifice, a buffer tank always keeps emptied, automatically resetting to attenuate the peak flow of runoff from nearby sealed surfaces for mitigating urban flooding.

![Figure 1. A buffer tank is set to temporarily storing runoff from a house’s rooftop](image)

### 2. Research Methodology

#### 2.1. Theory

As shown in Figure 1, runoff enters the buffer tank, in which a factor of runoff is detained and the surplus drains through the outlet orifice. According to the Torricelli Law, the outflow speed from the outlet orifice is proportional to the square root of the water head difference from the orifice to the water level in the tank, that is

\[ v = \sqrt{2gh} \]  

(1)

Where \( h \) (m\(^2\)) is the water head difference, \( g \) (m/s\(^2\)) is gravitational acceleration, \( g = 9.81 \). The corresponding outflow rate, \( q_o \) (m\(^3\)/s), is

\[ q_o = a\sqrt{2gh} \]  

(2)

Where \( a \) (m\(^2\)) is the cross-sectional area of the outlet orifice.

2526
Considering a rainfall intensity of $p$ (m/s) and a runoff coefficient of $r$ (-), the inflow to the buffer tank, $q_i$ (m³/s), is:

$$q_i = pAr$$

(3)

Where $A$ (m²) is the cross-sectional area of the sealed surfaces.

According to the law of conservation of mass, one has:

$$pAr - a\sqrt{2gh} = A_t \frac{dh}{dt}$$

(4)

Where $t$ (s) is time; $A_t$ (m²) is the internal cross-sectional area of the tank.

As both the precipitation $p$ and the water head difference $h$ are variables of time, it is difficult to find the analytic solution of $h$ in Equation 4. The practical solution is to find $h$ via the numerical differentiation. After finding the water head $h$, one can further find the water flowing rate $q_i$ and $q_o$, both of which are time series. The attenuation ratio, $\theta$ (-), of the peak flow can be estimated accordingly, which is

$$\theta = 1 - \max(q_o)/\max(q_i)$$

(5)

Where the operator $\max$ ($x$) finds the maximum value from a time series of $x$.

2.2. Experiments

We conducted indoor experiments to verify Equation 4. Two cubic tanks with an internal size of 0.5 x 0.5 x 0.5m were prepared (Figure 1). One tank was set at a high-lying position. The entire tank was watertight except a 6.0mm in diameter orifice at the tank bottom. Water discharging from this outlet was controlled by a faucet (Figure 2). The tip of the faucet was connected to a soft corrugated pipe to route the discharging water to the other tank, which was set at a lower position (Figure 2). Similarly, the low-lying tank was also watertight except that an orifice with 2.5mm in diameter was open at the tank’s bottom. Both tanks and the corrugated pipe were set to minimize the dynamical water circulation in the low-lying tank, and to ensure that water enters to the low-lying tank through gravitational flow. In this setup, the water discharging from the high-lying tank could be deemed as runoff from a sealed surface, such as a rooftop. The low-lying tank was the buffer tank that otherwise allows runoff discharging directly to the gutter without buffering. Except for the water volume detained in the tank, the surplus water in the low-lying tank drained from an orifice at the bottom of the buffer tank (Figure 2).

In the experiment, the high-lying tank was filled with water, while the low-lying one was empty. When the faucet was opened, water drained through the corrugated pipe to the low-lying tank. The faucet was controlled artificially. Several trials were repeated until the water outflow from the high-lying tank rose gradually from zero to a peak and then fell to zero, and until the receding limb was longer than the rising limb. The inflow and outflow from the buffer tank could be found by weighting the weight of the high- and low-lying tanks. However, considering that the dynamical water circulation in the tank varied the weight of the tank, the mass of the inflow and outflow water was not weighted directly. In addition, as the experiment lasted for hours, placing a weighting object on a scale for hours would increase the zero drift of the scale. Instead, the weight of water in both tanks was estimated through the water head in both tanks. The water head of each tank was measured using a water level meter in an interval of 1 min. After the high-lying tank was emptied, the reading interval was 2-4 mins because at this stage, the outflow of the low-lying tank was small.

![Figure 2. Experimental setups to observe the inflow and outflow from a buffer tank](image-url)

2527
3. Results

3.1. The Observed Water Head and the Water Flow Rate

In the experiment, only the water heads in high- and low-lying tanks are measured. These heads are used to derive the water-inflow and -outflow rates of the buffer tank. The inflow to the low-lying tank is estimated using the derivative of the water head in the high-lying tank with respect to time. The water storage in the low-tank is a derivative of the water head and is found by using the forward difference method. The outflow to the buffer tank is estimated using the Equation 2. As a result, the inflow fluctuates slightly. Ignoring these slight variations, the inflow rate gradually increases from zero to a peak flow of 57 ml/s, and then drops to zero. The inflow to the buffer tank lasts for about one hour. The hydrograph, as indicated in Figure 3a, with the rising limb shorter than the receding limb. The inflow to the buffer tank increases fast, which exceeds outflow for an hour (Figure 3a). As a result, the water head in the buffer tank rises gradually. The rise of water head continues until the inflow is equal to the outflow (Figure 3a and 3b).

![Figure 3. The observed water head and flow rate of the buffer tank (a) inflow and outflow rates of the buffer tank, (b) water head in the buffer tank](image-url)
While the water heads in high-lying and low-lying tanks are measured, only the water head in the low-lying (buffer) tank is of concern. It can be seen that the water head of the buffer tank rises to a peak about 0.33m. Recording that both tanks have the same size of 0.5m×0.5m×0.5m and that the high-lying tank is full of water before starting the experiment, the buffer tank needs to be 0.33m height only (dotted data in Figure 3b). Using this buffer tank, the peak flow is attenuated from 55ml/s to 13ml/s, which is equal to a peak-flow attenuation ratio of 76.4% (1-13/55). Correspondingly, the discharge from the buffer tank extends to 4.5 hours, which is about 3.5 hours longer than the rainfall duration. Considering that the intermission between two adjacent heavy rains is commonly larger than the rainfall duration, the use of the buffer tank is thus helpful to attenuate the peak flow of runoff.

The line plots in Figure 3 are the predicted data, which is estimated by using the observed inflow to the buffer tank as the input of Equation 4 to calculate the outflow and water head of the buffer tank. The predicted data (dotted data in Figure 3) is coincident well with the observed data, with a R² value greater than 0.95. The coincidence suggests that the discharge of water from the buffer tank can be deemed as an ideal fluid following the Torricelli Law. The coincidence is high when the water head is high (Figure 3). In case of a low water head, the observations deviate somewhat from the predictions because the viscous flow dominates in case of a small water flow. As the small discharge rate is not of concern in flooding, the outflow and the water head in the buffer tank can be predicted using the Torricelli Law.

3.2. The Buffer Tank Effectively Attenuates the Peak Flow in Case of Heavy Rain

The size of the buffer tank in the above experiment is relatively small. Here we simulate the use of a larger buffer tank to detain the runoff from a 100 m² sealed surface. We assume a rainfall depth of 100mm for an hour, which represents a typical heavy rain. We assume the instantaneous rainfall intensity as a gamma distribution, which is controlled by two constants α and β. Different α and β values are tried until the rainfall distribution curve, in shape, is similar to the inflow in Figure 3a. Finally, α = 2 and β = 500 are selected. According to this assumed rainfall intensity distribution and the mean rainfall, the peak instantaneous rainfall is 264.8 mm/hr. The runoff coefficient is assumed as 0.80. Using this assumed runoff as the input to Equation 4, a suite of simulations is conducted considering cubic buffer tanks with different outlet orifices. We found that if a cubic buffer tank has an internal edge length of 1.41m (√2) and an outlet orifice of 2.8cm in diameter, it can attenuate 45% of the peak flow (Figure 4a) and that the water head in the tank peaks at 1.41m (Figure 4b).

In a buffer tank, both the peak-flow attenuation and the peak water head are influenced by the outlet orifice. This influence is further simulated by using all the simulated factors in Figure 4 except the outlet orifice, which is set 2-4 cm. The simulation results showed that both the water head in the buffer tank and the attenuation of the peak flow decrease as the diameter of the outlet orifice increases (Figure 5). This correlation means that a large outlet orifice in the buffer tank requires a dwarf tank but results in a low peak-flow attenuation rate. This also means that an improperly large outlet orifice could make the tank losing the buffering capacity. Therefore, the outlet orifice and height of a buffer tank must be tailored such that the peak flow is attenuated to be lower than the carrying capacity of the local sewer pipe.
The water head and the flow rate of a buffer tank that is set to detain runoff from a 100m² rooftop subjected a rainfall depth of 10cm for an hour, (a) inflow and outflow of the buffer tank, (b) water heads in the buffer tank.

Either the water head in the buffer tank and the peak-flow attenuation decrease as the diameter of or outlet orifice increases.

3.3. The Buffer Tank Fails to Attenuate the Peak Flow in Case of Small Rain

There is a critical rainfall intensity in which the outflow of the buffer tank is equal to the inflow. As a result, the rainwater volume in the buffer tank is unchanged, that is, dh/dt=0. Substituting dh/dt=0 to Equation 4, one has:

\[ h = \frac{p^2 A^2 r^2}{2g} \]  

(6)

According to Equation 6, this critical water head, h, linearly increases with the square of the rainfall intensity, of a specific catchment area A, and of a constant runoff coefficient r. To verify this correlation, we simulate the water heads, outflow, and inflow of the buffer tank. The bottom area of the buffer tank is set as 2.0 m². The height of the buffer tank...
is assumed as 1.41m. Rainfall lasts for two hours. The rainfall intensity is assumed as a constant, which is sequentially set as 20, 40, 60, 80, 100 mm/hr in each simulation.

The simulation verifies that the water heads in the buffer tank increase with the square of the rainfall intensity, as indicated in Figure 6a; that is, the height of the buffer tank increases as the rainfall intensity increases. For a specific rainfall intensity, the water head in the buffer tank gradually rises first and then becomes a constant at a specific water head until the inflow is equal to the outflow (Figure 6a). The time reaching this balance increases as the rainfall intensity increases (Figure 6a), meaning that the buffer tank stores a large amount of runoff in case of heavy rain but a small volume in case of a small rain. As a result, a small long-lasting rainfall cannot fill the tank, which always preserves spaces for buffering runoff in case of a heavy rain.

![Figure 6](image_url)

Figure 6. Water head and flow rate of a buffer tank that is used to detain runoff from a 100m² sealed surface subjected to different rainfall intensity. (a) water head, (b) outflow and inflow, in which the dashed lines represent the outflow and straight lines, the inflow.
4. Discussion

We have shown that runoff from a 100 m² rooftop can be detained by a buffer tank for attenuating the peak flow about 20-70%. The volume of the buffer tank can be 2-3.5 m³, depending on the anticipated degree of peak-flow attenuation. In practice, the sealed surface can be greatly larger than 100 m², and thus a tank with a larger volume is required. The advantage of building a large tank is that the rainwater water can be treated (if necessary) collectively. However, as a buffer tank shall be buried at shallow ground for facilitating gravitational flow, a tank with larger volume means a greater base area for the tank, which required reinforced beams and columns to support the slabs that gap the tank. A large tank with this configuration will increases the construction cost. Therefore, in practice, rainwater runoff from a large sealed surface can be routed to a group of small tank, whose volume shall be designed according to the local site condition. Doing so, the urban flooding problem can be mitigated on sources. While the size and deployment of the buffer tanks, as well as the quality of water outflow from the tank, remain unknown, this study is starting point of the use of buffer tanks to mitigate urban flooding.

The tank can be set either above ground or underground to collect rainwater from the nearby catchment. As indicated in Figure 1, the tank can be set above ground to detain runoff from a rooftop. The discharge of water from the tank can be routed to a nearby gutter, where rainwater is diverted to local sewer pipes. The gutter still possibly overflows when the outflow from the buffer tank is large. Site-specific conditions need to be considered to avoid overflow by adopting a proper tank size and a proper outlet orifice. Another alternative is to bury the buffer tank underground and then to convert the outflow from the outlet orifice to the local sewer pipes. In this case, the runoff from the rooftop and other sealed surfaces (such as pavements) can be diverted to the buffer tank via gravitational flow. The outflow of the buffer tank also depends on the carrying capacity of the sewer pipes. If this capacity is lower than the instantaneous outflow of the buffer tank, the outflow is constrained and is equal to the carrying capacity. The true outflow and water head would be different from the simulation results in this study.

The buffer tank can be designed to attenuate the peak flow of runoff and to harvest rainwater simultaneously. The outlet orifice is not necessarily set at the bottom of the buffer tank. It can be placed at some elevations above the tank’s bottom to create a dead storage below the orifice, where the volume above the orifice is the live storage from attenuating the peak flow. Water stored in the dead storage can be extracted for non-portable water uses such as irrigations. In this scenario, the theory and results presenting in this study are only available to the live storage of the buffer tank. The height of the buffer tank and that of the outlet orifice need to be optimized to harvest an anticipated amount of rainwater and to attenuate a desirable peak flow rate. A buffer tank for this purpose may be beneficial of improving the quality of the outflow water. Water-carrying particles would settle at the bottom of the buffer tank because the water circulation rate in the tank is far lower than the speed of the inflow. Other contaminants may settle together with the settling particles at the bottom of the tank as well. Another setup of the buffer tank for harvesting rainwater and attenuating peak flow is illustrated in Figure 7. The tank is partitioned to two chambers. Water first filled the left chamber, where water is stored for reuse. After this chamber is filled, water automatically enters the right chamber, where a factor of water is detained and the remaining water drains from an outlet at the tank’s bottom. Further studies are needed to understand the performance of buffer tanks for attenuating the peak flow of runoff and harvesting rainwater.

Figure 7. The buffer tank can be set to attenuate runoff and harvest rainwater simultaneously. (a) The outlet orifice is set at an elevation from the tank bottom to create a dead storage for harvesting rainwater, (b) the tank is partitioned to a dead space for harvesting rainwater and a live storage for attenuating runoff
5. Conclusion

This study proposes a novel-rainwater buffering tank to store rainwater for reducing the peak-flow discharge of the runoff from the nearby sealed surface. Runoff is converted to the tank and is then discharged from an outlet orifice that sets at the tank’s bottom. In case of a small rain, the outflow could be equal to the inflow such that the tank fails to attenuate the peak flow. In case of a heavy rain, the inflow is greater than the outflow, with a part of rainwater being detained in the buffer tank and with the surplus discharging from the outlet orifice. After rainfall stops, the water detained in the buffer tank drains automatically from the outlet orifice. The buffer tank is thus always emptied to temporarily detain runoff in case of a heavy rain.

A theoretical model on the basis of the Torricelli Law is developed to calculate the water head in the buffer tank and to estimate the discharging rate. Experimental observations confirm that the outflow from the buffer tank obeys the Torricelli Law. Considering a 100 m² rooftop with 0.8 runoff coefficient subjected to 100mm for an hour, a cubic tank with internal edge side of a square of 2.0 m attenuates the peak flow about 45%. The attenuation rate decreases as the outlet orifice increases. The orifice thus must be tailored to attenuate the peak flow according to site-specific conditions. In practice, the buffer tank can be set above ground to store runoff from rooftops, or be buried underground to collect runoff from rooftops and other sealed surfaces. The outflow from a buried buffer tank needs to be further studied in case that the outflow exceeds the carrying capacity of the local sewer pipes. In addition, the tank can be designed to attenuate the peak flow of runoff and harvest rainwater simultaneously.

6. Funding

This work is supported by Natural Science Foundation of China (Grant Nos. 41561015, 51508114) and by the Science Foundation of Guangxi (Grant no. 2018GXNSFAA294070).

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Moisture Susceptibility of Asphalt Concrete Pavement Modified by Nanoclay Additive

Saif Al-din Majid Ismael a*, Mohammed Qadir Ismael b

a M.Sc. Student, Civil Engineering Department, University of Baghdad, Baghdad, Iraq.
b Assistant Professor, Civil Engineering Department, University of Baghdad, Baghdad, Iraq.

Received 06 August 2019; Accepted 18 October 2019

Abstract

Durability of hot mix asphalt (HMA) against moisture damage is mostly related to asphalt-aggregate adhesion. The objective of this work is to find the effect of nanoclay with montmorillonite (MMT) on Marshall properties and moisture susceptibility of asphalt mixture. Two types of asphalt cement, AC(40-50) and AC(60-70) were modified with 2%, 4% and 6% of Iraqi nanoclay with montmorillonite. The Marshall properties, Tensile strength ratio (TSR) and Index of retained strength (ISR) were determined in this work. The total number of specimens was 216 and the optimum asphalt content was 4.91% and 5% for asphalt cement (40-50) and (60-70) respectively. The results showed that the modification of asphalt cement with MMT led to increase Marshall stability and the addition of 6% of MMT recorded the highest increase, where it increased by 26.35% and 22.26% foe asphalt cement (40-50) and (60-70) respectively. Also, the addition of MMT led to increase moisture resistance of asphalt mixture according to the increase in TSR and IRS. The addition of 4% and 6% of MMT recorded the highest increase in TSR and IRS for asphalt cement (40-50) and (60-70) respectively, where they increased by 11.8% and 17.5% respectively for asphalt cement (40-50) and by 10% and 18% respectively for asphalt cement (60-70).

Keywords: Moisture Susceptibility; Nanoclay; Montmorillonite (MMT); Tensile Strength Ratio (TSR); Index of Retained Strength (ISR).

1. Introduction

Moisture damage is a failure mode that have an effect on a great number of pavements around the world. The damages of moisture can be considered as a distresses that affect on society in different ways. Economically, moisture damage costs millions of dollars for maintenance and rehabilitation of the damaged pavements [1, 2]. In asphalt pavements which are exposed to moisture infiltration, detachment of the aggregate from the mix is usually the main problem [3]. The continuation the action of moisture with high traffic load cause weakening in the mechanical properties of asphalt mixture which causes a gradual dislocation of the aggregate and asphalt, in some cases, this type of damage becomes a Prevailing failure and a cause for diminish road safety [4]. The type of this damage is known as stripping or ravelling of the wearing layer of asphalt mixture. The damage caused by initial stripping can rapidly develop into a more intense disintegration of the wearing surface, and lastly lead to pothole formation as shown in Figure 1 [5]. The moisture damage in asphalt pavement can cause two types of failures: loss of asphalt-aggregate adhesion and loss of cohesion within the asphalt binder [6]. Figure 2 show the failures caused by moisture damage.

The use of nano-materials in asphalt mixtures presents one of the most attractive options [7]. According to the

*Corresponding author: saifmajid363@gmail.com

doi: http://dx.doi.org/10.28991/cej-2019-03091431

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
European Union Recommendation 2011/696/EU, nanomaterials are materials with at least one dimension in the size range 1nm–100 nm or from 10−9 up to 10−7m [8]. Due to high specific area, nano-materials (if properly mixed) establish a strong network in asphalt and therefore lead to better mechanical properties [9].

![Image](image1.jpg)

Figure 1. Moisture Damage in Asphalt Pavement [2]

![Image](image2.jpg)

Figure 2. Stripping due to cohesive and adhesive failures in the asphalt wearing surface

Modifiers increases construction costs of the pavements significantly [10]. In order to overcome these shortcomings, researchers have recently suggested the use of nanoclay (NC) with montmorillonite (MMT) as asphalt modifiers due to their abundance in nature, low cost of production, and small amounts needed for asphalt modification [11]. Nanoclay is one of the most common, safe, inexpensive, and sustainable layered silicate, with montmorillonite (MMT) being the most using. It is one of the newest additives being used in asphalt mixtures to increase their performance [12]. The cost of MMT nanoclay modified asphalt binder can be approximately 22-33 % lower than of polymer modified asphalt binder [13]. Modified asphalt binder by MMT is an effective way to enhance moisture resistance of asphalt mixture [14]. As contact surface of MMT nanoclay is extremely high, blending it with asphalt results in greater interaction between aggregate and asphalt [15]. Bentonite is an important source of montmorillonite (MMT) in nature [16]. Montmorillonite is one of the types of nanoclay clay that have layer structure containing SiO₂ and Al₂O₃ with various configuration to form a layer, SiO₂ to Al₂O₃ ratio of 2:1 [15]. The layered structure of MMT is shown in Figure 3 and 4.

![Image](image3.jpg)

Figure 3. The structure of Montmorillonite
Ameli et al., 2016 noticed that the penetration and ductility continued in decrease with addition of (10, 15, 20, 25 and 30)% of MMT by weight of asphalt cement grade 60/70, where the penetration decreased from 68 to 11 and the ductility decreased from 100 cm to 25 cm when MMT content changed from 0 to 30%. The addition of 30% of MMT also recorded the highest increase in softening point, where it increased from 48 to 61 °C [17].

Babagoli et al., 2017 showed that the addition of (2, 3, 4 and 5)% of MMT by weight of asphalt cement grade 60/70 led to decrease the penetration and ductility and increase softening point. The highest decrease in penetration and ductility was at 5% content of MMT, where the penetration decreased from 64 to 50 and ductility decreased from 110 cm to 105 cm. at 4% content of MMT the softening point was had the highest increase, where it increased 53 to 64 °C [18].

Jahromi and Rajaee, 2013 notice the increase in stability and voids in total mixture percent with addition of 2, 4 and 7 % of MMT, where Marshall stability increased by 6 % when 2 % of MMT was added, while 7 % content of MMT recorded the highest value for each stability and air voids. Because of high surface area for MMT, the optimum content for modified asphalt with MMT increased by 0.3-0.35% compared with unmodified asphalt mixture [19].

Iskender, 2016 studied the effect of MMT on Marshall properties of asphalt mixture. The flow, air voids and voids in mineral aggregate decreased with addition of 2, 3.5 and 5 % of MMT, and the addition of 2 % of MMT recorded the highest decrease by 16.22, 33.33 and 4.55 % respectively compared with control mixture. Marshall stability also recorded the highest improvement at 2% content of MMT, where it increased by 20%. The density of asphalt mixture and voids filled with asphalt increased by 1.23 and 9.8 % respectively with addition of 2 % of MMT [20].

Zahedi and Baharvand, 2017 studied the effect of addition of MMT with another types of additives on the asphalt mixture. (1, 2, 3, 4 and 5)% of MMT and (5, 10, 15, 20 and 25)% of crumb rubber were mixed with asphalt cement grade 60/70. Addition of 10% crumb rubber gave the higher increase in Marshall stability up to 40% with different percentages of MMT against the control samples. Total mixture voids has increased about 36% against the control sample by increasing crumb rubber and MMT portions in the mixture which could be a result of chemical interaction between MMT and crumb rubber in the asphaltic mixture. The voids in mineral aggregates increased by 9% by rising the portion of MMT and crumb rubber in the asphaltic mixture. The voids filled with asphalt has decreased up to 8% against the control sample by increasing MMT and crumb rubber portions. Special gravity has decreased 2% against control samples by increasing MMT and crumb rubber portions in asphaltic mixtures [21].

Omar et al., 2018 showed that the addition of 4 % of MMT led to increase the optimum asphalt content and voids filled with asphalt by 5.36 and 5.7% respectively compared with control mixture which recorded 5.18 and 74.1 %. Also the air voids and voids in mineral aggregate was 4.92 and 17.1 % for control mixture and the addition of the same percentage of MMT led to increase it by 11 and 8.9 %. The addition of the same percent of MMT led to increase the dry indirect tensile strength (ITS) 3.9 to 4.2 kN, while the wet ITS increased from 2.6 to 3.2 kN and as a result of this, TSR increased from 75 to 79 % [22].

Malarvizhi et al., 2015 mixed (5.5, 6 and 6.5) % of asphalt cement grade 60/70 with (1, 2, 3and 4) % of MMT and noticed its effect on the moisture susceptibility of asphalt mixture depending on tensile strength ratio (TSR) test. TSR increased with increase of asphalt content from 5.5 to 6 % irrespective the content of MMT, then it decreased at 6.5% of asphalt content. The TSR value reached the highest value at 3% of MMT content, where it increased from 94 to 96 % when the bitumen content increased from 5.5 to 6 %, and it decreased to 93 at 6.5 % asphalt content. 2% of MMT improved the TSR from 88 to 91 % when the asphalt content changed from 5.5 to 6 % and, then it decreased to 90 at 6.5% asphalt content. When 4% of MMT was added, the TSR value increased from 81 to 93 % with increase of asphalt content from 5.5 to 6 % and it return to decrease at 6.5% asphalt content, where it was 91%. Finally, the TSR also increased when 1% of MMT was added, where TSR increased from 83% to 94% when asphalt content increased from 5.5 to 6 % and also it decreased to 90 at 6.5% asphalt content [23].
De Melo et al., 2017 showed that the addition of 3% of MMT by weight of asphalt led to increase moisture resistance of asphalt mixture depending on the increase in TSR value by 10% compared with control mixture, where it increased from 91 to 100.5 % [24].

Amini et al. (2017) notice the increase in TSR by 9 and 13% with addition of 2.5 and 5% of MMT by weight of asphalt respectively [25].

Othman et al., 2017 added 2 and 4 % of MMT by weight of asphalt cement grade (60-70) and studied the effect of these addition on moisture susceptibility of asphalt mixture depending on TSR test. The dry indirect tensile strength (ITS) do not affected when 2% of MMT was added, while the wet ITS increased. At 4 % content of MMT, both dry and wet ITS were increased. TSR increased by 22 and 23% at 2 and 4% of MMT content respectively [26].

The primary objective of this work is to evaluate the effect of using Iraqi nanoclay with montmorillonite (MMT) on Marshall properties and moisture susceptibility of wearing course of asphalt pavement and compare it with control mixture by using two types of asphalt cement, AC (40-50) and AC (60-70) considering the values of the Tensile Strength ratio (TSR) and Index of Retained Strength (IRS).

2. Research Methodology

The program of this work can be seen in by a flow chart as shown in Figure 5.

![Flow Chart for Research Methodology](image-url)
3. Materials and Methods

The research methodology of this work was involved four stages, the first stage included the bringing of materials (asphalt cement, fine and coarse aggregate, mineral filler and Iraqi clay with montmorillonite) and testing the physical properties for these materials. The second stage covered the process of grinding MMT to the nano-size and analyzing its chemical and mineral composition. The third stage covered the design of the asphalt mixture with and without addition of MMT by using Marshall method and obtaining the optimum asphalt content for each percent of MMT. The fourth stage included the evaluation of indirect tensile strength to find the tensile strength ratio (TSR) and compressive strength to find the Index of retained strength (IRS) for each percent of MMT to find best percent of MMT that give the highest resistance to moisture damage.

3.1. Asphalt Cement

Two types of asphalt cement were used in this work, AC (40-50) and AC (60-70) penetration grade which were obtained from Al-Daurah refinery. The Physical properties of these two types of asphalt cement are listed in Tables 1 and 2. All tests results meet the Stat Corporation for Roads and Bridges (SCRB R/9, 2003) specification.

<table>
<thead>
<tr>
<th>Test</th>
<th>Result</th>
<th>SCRB Specification Limits [27]</th>
<th>ASTM Designation No. [28]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (25°C, 100 gm, and 5sec)</td>
<td>44 (0.1mm)</td>
<td>40 - 50</td>
<td>D-5</td>
</tr>
<tr>
<td>Softening point (Ring and Ball)</td>
<td>51 (° C)</td>
<td>------</td>
<td>D-36</td>
</tr>
<tr>
<td>Ductility, (25 ° C and 5cm/minute)</td>
<td>162 (cm)</td>
<td>≥ 100</td>
<td>D-113</td>
</tr>
<tr>
<td>Specific Gravity @ 25 ° C</td>
<td>1.042</td>
<td>------</td>
<td>D-70</td>
</tr>
<tr>
<td>Flash point ,(Cleveland open Cup)</td>
<td>309 (° C)</td>
<td>&gt;232</td>
<td>D-92</td>
</tr>
</tbody>
</table>

After Thin-Film Oven Test

<table>
<thead>
<tr>
<th>Tests</th>
<th>Result</th>
<th>SCRB Specification Limits [27]</th>
<th>ASTM Designation No. [28]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retained penetration, of original, (%)</td>
<td>61(0.1mm)</td>
<td>&gt;55</td>
<td>D-5</td>
</tr>
<tr>
<td>Ductility (25°C and 5cm/minute)</td>
<td>89 (cm)</td>
<td>&gt;25</td>
<td>D-11</td>
</tr>
</tbody>
</table>

Table 2. Physical properties of asphalt cement grade (60-70)

<table>
<thead>
<tr>
<th>Test</th>
<th>Result</th>
<th>SCRB Specification Limits [27]</th>
<th>ASTM Designation No. [28]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (25°C, 100 gm, and 5sec)</td>
<td>67 (0.1mm)</td>
<td>60 - 70</td>
<td>D-5</td>
</tr>
<tr>
<td>Softening point (Ring and Ball)</td>
<td>48 (° C)</td>
<td>------</td>
<td>D-36</td>
</tr>
<tr>
<td>Ductility, (25 ° C and 5cm/minute)</td>
<td>149 (cm)</td>
<td>≥ 100</td>
<td>D-113</td>
</tr>
<tr>
<td>Specific Gravity @ 25 ° C</td>
<td>1.021</td>
<td>------</td>
<td>D-70</td>
</tr>
<tr>
<td>Flash point ,(Cleveland open Cup)</td>
<td>285 (° C)</td>
<td>&gt;232</td>
<td>D-92</td>
</tr>
</tbody>
</table>

After Thin-Film Oven Test

<table>
<thead>
<tr>
<th>Tests</th>
<th>Result</th>
<th>SCRB Specification Limits [27]</th>
<th>ASTM Designation No. [28]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retained penetration, of original, (%)</td>
<td>63(0.1mm)</td>
<td>&gt;55</td>
<td>D-5</td>
</tr>
<tr>
<td>Ductility (25°C and 5cm/minute)</td>
<td>82 (cm)</td>
<td>&gt;25</td>
<td>D-11</td>
</tr>
</tbody>
</table>

3.2. Coarse and Fine Aggregates

The crushed coarse aggregate was brought from the hot mix plant of Baghdad municipality, where the source of aggregate was from Al-Nibae quarry. The fine aggregate consists of hard grains, free of injurious amount of clay, loam or other deleterious substances. It was also brought from the hot mix plant of municipality Baghdad. According to (SCRB R/9, 2003) specification [27], the sizes of coarse aggregate ranged from 3/4 in (19 mm) to No.4 sieve (4.75mm) for wearing course and the fine aggregate is ranged from passing sieve No.4 (4.75mm) to the retained on sieve No.200 (0.075 mm). The physical properties of coarse and fine aggregate are listed in Table 3.
3.3. Mineral Filler

Limestone dust was used as a filler in preparing the asphalt mixture due to its availability and relatively lower cost. The mineral filler in asphalt mixture is an important component of the mixture as it plays an important role in stiffening and toughening an asphalt binder. Filler is a material passing sieve No.200 (0.075 mm). It was brought from lime factory in Karbala governorate. The physical properties of the mineral filler are listed in the Table 4.

Table 4. Physical properties of limestone dust

<table>
<thead>
<tr>
<th>Property</th>
<th>Limestone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Specific Gravity</td>
<td>2.67</td>
</tr>
<tr>
<td>% Passing No.200</td>
<td>99</td>
</tr>
</tbody>
</table>

3.4. Nanoclay (NC)

Iraqi naoclay with montmorillonite (MMT) which is locally and commercially known as bentonite as shown in Figure 6 was used in this work to enhance the moisture resistance of asphalt mixture, where many scientific researches indicated that the use of nanoclay is useful for this purpose. It was brought from department of geological survey in Baghdad governorate which is supplied with this type of clay from the Bentonite quarry in Al-fallujah city. The Iraqi clay which is sending from the quarry to the department of geological survey is not in nanosize. Therefore, processes of grinding were needed to get nanosize for this clay before adding it to the asphalt cement.

Figure 6. Iraqi bentonite

3.4.1. Grinding of Bentonite (Montmorillonite) to the Nano-size

2 kg of bentonite was brought and divided into 8 equal parts (250 gm for each part). Each part was sieved on sieve No.50, the retaining on this sieve was left and the passing from it was taken to complete new grinding processes. After that, 1 kg of bentonite was taken from passing sieve no.50 and divided into 10 equal parts (100 gm for each part) as shown in Figure 7-A. One of these parts was taken and subjected to successive grinding processes. The sample was placed in an electric mill and it was operated for 10 minutes intermittently with continuous rotation during operation to distribute the milling process on the entire sample as shown in Figure 7-B. The sample was extracted from an electric mill, placed inside a nylon bag, brushed to the largest possible area and knocked by hand hammer continuously for 5 minutes as shown in Figure 7-C. Then it was returned to the electric mill to repeat the grinding process. The processes of grinding in a mill and brushed inside a nylon bag and knocked by hand hammer were repeated six times, then the sample was brushed inside a circular dish with a diameter of 25 cm and it was crushed in circular motion by a thick spoon for 10 minutes as shown in Figure 7-D and returned to the electric mill to repeat the grinding process for last time (seventh time). After completing the grinding process and reaching to a very high degree of smoothness of bentonite,
the particle size was examined by a laser device to determine if the bentonite reached the nanosize or not. The shape and particles size of bentonite under a laser device are shown in Figures 8 and 9 respectively.

The result of this test in figure showed that the average diameter of particles size of bentonite is 115.53 nm and this does not meet the requirement for nanometer measurements, where according to the European Union Recommendation 2011/696/EU, nanomaterials are materials with at least one dimension in the size range 1nm–100 nm or from $10^{-9}$ up to $10^{-7}$ m [8]. Therefore, another steps were needed to increase the grinding process and to reach to the limits of nanometer.

![A B C D](image)

Figure 7. The Initial Grinding Processes of Bentonite

![Image](image)

Figure 8. The Shape of Particle Size of Bentonite under the Laser Device after Initial Grinding Processes
A larger electric mill with a very high specification that can be used to break the gravel was brought. The rotation speed of this mill is very high (25000 rpm). The bentonite which has average diameter 115.53 nm was placed in this electric mill and it was operated for 25 minutes intermittently with continuous rotation during operation. The properties of the mill was used in this step are shown in Figure 10.

Finally, sample was examined secondly by the same device to determine if the bentonite reached the nanosize or not. The size and shape of particles of bentonite under the laser device after the second grinding process are shown in Figures 11 and 12 respectively. The result showed that the average diameter of particles size was 75.12 nm and this does meet the requirement for nanometer measurements.
3.4.2. X-ray Fluorescence (XRF) and X-ray Diffraction (XRD) Tests

The chemical composition and mineral analysis of bentonite were performed by XRF and XRD to know the properties of Iraqi bentonite. X-ray diffraction (XRD) spectra, which are used to specify distance between bentonite layers, were obtained using copper (Cu-κα) radiation, scanning from 1.5° to 15° in the 2θ range in 0.01° steps at a scanning rate of 2°/min. The distance was calculated by applying Bragg’s law according to the equation below [24].

\[ n\lambda = 2dsin\theta \]  

Where,
\( n \) = Order of diffraction;
\( \lambda \) = Radiation wavelength of the X-ray used in the experiment (1.5406 Å);
\( d \) = Spacing between the planes of the diffraction grating;
\( \theta \) = Diffraction angle measured.

The chemical composition of Iraqi bentonite is shown in Table 5 and the mineral analysis and its explanation are shown in Figure 14.
Table 5. The chemical composition of Iraqi Bentonite

<table>
<thead>
<tr>
<th>Z</th>
<th>Symbol</th>
<th>Element</th>
<th>Norm. Int</th>
<th>Concentration</th>
<th>Abs. Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>MgO</td>
<td>Magnesium</td>
<td>95.0336</td>
<td>2.436 %</td>
<td>0.024 %</td>
</tr>
<tr>
<td>13</td>
<td>Al₂O₃</td>
<td>Aluminum</td>
<td>1714.8835</td>
<td>10.65 %</td>
<td>0.02 %</td>
</tr>
<tr>
<td>14</td>
<td>SiO₂</td>
<td>Silicon</td>
<td>16660.5205</td>
<td>42.87 %</td>
<td>0.03 %</td>
</tr>
<tr>
<td>15</td>
<td>P₂O₅</td>
<td>Phosphorus</td>
<td>650.1428</td>
<td>0.9263 %</td>
<td>0.002 %</td>
</tr>
<tr>
<td>16</td>
<td>SO₃</td>
<td>Sulfur</td>
<td>1613.7953</td>
<td>1.192 %</td>
<td>0.002 %</td>
</tr>
<tr>
<td>17</td>
<td>Cl</td>
<td>Chlorine</td>
<td>494.0189</td>
<td>0.05992 %</td>
<td>0.00016 %</td>
</tr>
<tr>
<td>19</td>
<td>K₂O</td>
<td>Potassium</td>
<td>113.4929</td>
<td>0.4869 %</td>
<td>0.0035 %</td>
</tr>
<tr>
<td>20</td>
<td>CaO</td>
<td>Calcium</td>
<td>2099.7205</td>
<td>7.474 %</td>
<td>0.010 %</td>
</tr>
<tr>
<td>26</td>
<td>Fe₂O₃</td>
<td>Iron</td>
<td>16045.2853</td>
<td>5.492 %</td>
<td>0.004 %</td>
</tr>
</tbody>
</table>

4. Selection of Aggregates Gradation

According to (SCRB/R9, 2003) specification, the nominal maximum size of aggregate is 12.5 mm for wearing course. The selected aggregate gradation is shown in Figure 13.

![Gradation of the aggregates for wearing course according to SCRB](image)

Figure 13. Gradation of the aggregates for wearing course according to SCRB [22]
5. Incorporating of Bentonite (MMT) with Asphalt

Asphalt cement has been heated to a 150 °C and then blended with a bentonite (MMT) with different percentages (2, 4 and 6) % by weight of asphalt cement on a hotplate at a blending speed of about 1500 rpm for 40 minutes. After that, five percentages (4, 4.5, 5, 5.5 and 6%) of asphalt cement by weight of total mix were utilized to find the optimum asphalt content for the HMA for two types of asphalt cement, and three specimens were made for each percent of asphalt. The specimens were subjected to a series of Marshall tests, stability, flow, and density-voids analysis to determine the optimum asphalt content by using (12.5 mm nominal maximum size gradation of aggregate) and 7% of lime stone dust as mineral filler (by weight of total aggregate). The average of each set (three specimens for each asphalt percent) was obtained, then the same procedure was returned for asphalt mixtures modified by 2, 4 and 6 %.

6. Tensile Strength Ratio Test (TSR)

This test method can be used to evaluate the effect of moisture on the asphalt mixture with and without anti stripping additives. This test is fully covered by ASTM D- 4867. At first, four Marshall specimens 4 inch (101.1 mm) in diameter and 2.5 inch (63.5 mm) in height without any additive conforming to the job-mix formula are prepared by trial method with (40, 50, 60 and 70) blows respectively to get the number of blows that give 7±1% air voids. After determining the number of blows, four set of Marshal specimens were prepared with six specimens for each set. The first set was without any additive and the other three set were modified with (2, 4 and 6) % of bentonite respectively. After that, each set was divided to two groups (three specimens for each group). The first group was placed in water bath at 25 °C for 30 minutes (unconditioned samples) and the indirect tensile strength (ITS) was calculated for each specimen and the average ITS was calculated for three specimens. The second group was placed at a vacuum container filled with distilled water at 25 °C to remove the air content. After that it was placed in the freezer at a temperature of -18°C for a 16 hours. After the
freezing cycle, the thawing cycle begun by placing the specimens at a water bath at 60 °C for 24 hours. Then, it was extracted and placed at another water bath at 25 °C for 1 hour and the ITS was calculate for these (conditioned samples). The calculations of indirect tensile strength (ITS) are determined according to the method described by (ASTM D-6931-12). The (ASTM D-4867-09) specified that the minimum TSR value is 80%

\[
I.T.S = \frac{2000P}{\pi.t.D}
\]

\[
TSR = \frac{\text{Con.ITS}}{\text{Uncon.ITS}}
\]

Where,

P: ultimate applied load required to fail specimen (N);

\(t\): thickness of the specimen (mm);

D: diameter of specimen (mm);

Con. ITS = Conditioned indirect tensile stress (kPa);

Uncon. ITS = Unconditioned indirect tensile stress (kPa).

7. Compressive Strength Test

This test method is useful as an indicator of the susceptibility to moisture of compacted bitumen-aggregate mixtures and fully covered by ASTM D-1075-07. According to ASTM D-1074-02, four set of cylindrical specimens, 4 inch in height and 4 inch in diameter (101.6×101.6 mm) were prepared by compressing the asphalt mixture until the specimen reaches the required height (101.6 mm). The first set was without any additive for asphalt cement while the other three set were with modification of asphalt cement with 2, 4 and 6% of bentonite respectively and each set consist of 6 specimens. Three specimens from each set were placed in air bath at 25 °C for about 4 hours (dry specimens) and after that tested for compressive strength. The other three specimens from each set were immersed in a water bath for 24 hours at 60 °C (immersed specimens) then, they were extracted and placed in another water bath at 25 °C for 2 hours before testing for compressive strength. The index of retained strength (IRS) was calculated According to ASTM D-1075-07 and according to SCRB/R9, 2003, the minimum value of IRS should be 70%.

\[
IRS = \left(\frac{S2}{S1}\right) \times 100
\]

Where,

S1: Compressive strength of dry specimen (kpa);

S2: Compressive strength of immersed specimen (kpa).

8. Results and Discussion

8.1. Marshall Tests

The results showed that the optimum asphalt content increased with increase of MMT content, where it increased by 4.28, 8.55 and 11.6% for asphalt cement AC(40-50) and by 2, 5 and 6.6% for asphalt cement AC(60-70) with addition of 2, 4 and 6% of MMT respectively compared with control mixture which was 4.91 and 5%.

The increase in asphalt content lead to increase in bulk density of asphalt mixture. Then, after reaching to the certain percentage of increasing, the bulk density begin decreasing because the high percentage of asphalt trends to displace the aggregate outside the mixture. The bulk density increased by 0.73, 0.91 and 1.17% for asphalt cement AC (40-50) and
by 0.6, 0.63 and 0.48% for asphalt cement AC (60-70) with addition of 2, 4 and 6% of MMT respectively compared with control mixture which was 2.306 gm/cm³ and 2.299 gm/cm³. Also due to high surface area for MMT nanoclay added to the asphalt, the asphalt mixture become denser which led to this increase in bulk density.

The increase in viscosity and softening point of asphalt cement and also the increase in bulk density of asphalt mixture with addition of MMT led to increase Marshall stability of asphalt mixture, where it increased by 5.6, 19.9 and 26.35% for asphalt cement AC (40-50) and by 15.26, 18.8 and 22.26% for asphalt cement AC (60-70) with addition of 2, 4 and 6% of MMT respectively compared with control mixture which was 9.45 and 7.93 kN. Marshall stability is one of the most important indication to the resistance of asphalt pavement to the traffic loading, where the high stability means a highest stiffness for asphalt mixture.

The high flow gives an indication that the asphalt mixture have a low resistance under the effect of traffic loading. A high content of asphalt cement lead to increase in Marshall flow. The results showed that the Marshall flow decreased by 5.76 and 17.3% for asphalt cement AC(40-50) with addition of 2 and 4% of MMT respectively despite of increase in asphalt content. At 6% content of MMT, the flow returned to increase but it remained in an decreasing state compared with control mixture which was 3.47 mm, where it decreased by 4.6%. For asphalt cement AC(60-70), Marshall flow decreased by 7.1 and 4.7% with addition of 2 and 4% of MMT respectively, but it increased by 1.9 at 6% content of MMT compared with control mixture which recorded 3.64 mm.

The voids in total mixture is important parameter that an effect on the durability of asphalt mixture. According to SCRB (R/9), 3-5% of AV is enough to prevent bleeding which occur as a result of low AV (less than 3%) in asphalt mixture, while the high AV (more than 5%) make the asphalt mixture less durable against the effect of moisture and fatigue. The modification of asphalt cement AC (40-50) by 2, 4 and 6% of MMT led to decrease the AV % by 2.37, 3.16 and 14.2% respectively. For asphalt cement AC (60-70), AV decreased by 3.65, 6.32 and 4.71% when 2, 4 and 6% of MMT were added respectively compared with control mixture. As a result of high surface area and high degree of smoothness for MMT nanoclay added to the asphalt cement and increase in the bulk density of asphalt mixture, the AV % recorded low values compared with control mixture.

The voids between the aggregate particles which known as voids in mineral aggregate are one of the important parameters that should be high enough to get good coating for aggregate by asphalt and reduce the loss of adhesion between asphalt and aggregate which lead to increase the durability of asphalt mixture. The addition of 2, 4 and 6% of MMT for asphalt cement AC (40-50) led to decrease the VMA % by 0.5, 0.12 and 0.87% respectively. For asphalt cement AC (60-70), the addition of the same percentages of MMT led to decrease the VMA % by 2.96, 2 and 0.65% with addition of 2, 4 and 6% of MMT respectively compared with control mixture.

All results of Marshall properties comply with the results of most former researchers, where the increase in bulk density comply with (İskender, 2016) and the increase in Marshall stability, decrease in Marshall flow, decrease in air voids, decrease in voids in mineral aggregate and increase in voids filled with asphalt comply with (İskender, 2016) and (Omar et al., 2018).

### Table 6. Marshall Test Results for Asphalt Cement Grade (40-50)

<table>
<thead>
<tr>
<th>MMT, (%) by wt. of Asphalt</th>
<th>O.A.C., (%) by wt. of mix.</th>
<th>Stability (kN)</th>
<th>Flow (mm)</th>
<th>Bulk Density (gm/cm³)</th>
<th>Air Voids (%)</th>
<th>V.M.A (%)</th>
<th>V.F.A (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>4.91</td>
<td>9.45</td>
<td>3.47</td>
<td>2.306</td>
<td>3.8</td>
<td>16.18</td>
<td>76.44</td>
</tr>
<tr>
<td>2</td>
<td>5.12</td>
<td>9.98</td>
<td>3.27</td>
<td>2.323</td>
<td>3.71</td>
<td>16.1</td>
<td>76.95</td>
</tr>
<tr>
<td>4</td>
<td>5.33</td>
<td>11.33</td>
<td>2.87</td>
<td>2.327</td>
<td>3.68</td>
<td>16.16</td>
<td>77.17</td>
</tr>
<tr>
<td>6</td>
<td>5.48</td>
<td>11.94</td>
<td>3.31</td>
<td>2.333</td>
<td>3.26</td>
<td>16.04</td>
<td>79.68</td>
</tr>
</tbody>
</table>

### Table 7. Marshall Test Results for Asphalt Cement Grade (60-70)

<table>
<thead>
<tr>
<th>MMT, (%) by wt. of Asphalt</th>
<th>O.A.C., (%) by wt. of mix.</th>
<th>Stability (kN)</th>
<th>Flow (mm)</th>
<th>Bulk Density (gm/cm³)</th>
<th>Air Voids (%)</th>
<th>V.M.A (%)</th>
<th>V.F.A (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5</td>
<td>7.93</td>
<td>3.64</td>
<td>2.299</td>
<td>4.334</td>
<td>16.91</td>
<td>74.37</td>
</tr>
<tr>
<td>2</td>
<td>5.12</td>
<td>9.14</td>
<td>3.38</td>
<td>2.313</td>
<td>4.176</td>
<td>16.41</td>
<td>74.55</td>
</tr>
<tr>
<td>4</td>
<td>5.25</td>
<td>9.42</td>
<td>3.47</td>
<td>2.3135</td>
<td>4.06</td>
<td>16.57</td>
<td>75.48</td>
</tr>
<tr>
<td>6</td>
<td>5.33</td>
<td>10.14</td>
<td>3.71</td>
<td>2.31</td>
<td>4.13</td>
<td>16.8</td>
<td>75.5</td>
</tr>
</tbody>
</table>
8.2. Indirect Tensile Strength Test Results

The dry ITS for asphalt cement AC (40-50) increased with increase of MMT content, where it increased by 11.51 and 22.33% with addition of 2 and 4% of MMT respectively. This increase in dry ITS decreased at 6% of MMT content, where it increased by 13.82% compared with control mixture. For asphalt cement AC (60-70), the dry ITS increased by (18.45, 24.64 and 37.87)% with addition of (2, 4 and 6)% of MMT respectively. The wet ITS for asphalt cement grade (40-50) increased by 15.33 and 36.77 % when 2 and 4 % of MMT were added respectively and it increased by 21.31% at 6% of MMT content. For asphalt cement grade (60-70), the wet ITS increased in high percentages with addition of (2, 4 and 6)% of MMT, where it increased by (18.45, 24.64 and 37.87)% respectively.

Because of high surface area of MMT Nanoclay, the aggregate was coated by enough content of asphalt and as a result of this, the adhesion force between asphalt and aggregate became higher which prevent stripping under the effect of moisture. This increased in adhesion force led to increase the (ITS) in both dry and wet condition, but the increase in wet condition was higher which mean that the asphalt mixture loses a little of its strength under the effect of moisture with addition of MMT to the asphalt cement. Also there are several negative charge ions on the surface of particles of MMT. However, the presence of positive ions between these particles and asphalt, brings about cohesion and adhesion between them. The crystal structures of clay minerals have given this category of nanomaterials a good efficiency rating to be applied for this purpose. As a result of this, the tensile strength ratio (TSR) increased and this increase in TSR give an indication to the increase the moisture resistance of asphalt mixture with addition of MMT. The effect of MMT on ITS in dry and wet conditions are shown in Figures 17 and 18 respectively.

![Figure 17. Effect of MMT on Indirect Tensile Strength for asphalt cement AC (40-50)](image1)

![Figure 18. Effect of MMT on Indirect Tensile Strength for asphalt cement AC (60-70)](image2)

According to the increase in ITS for dry and wet condition, the Tensile strength ration (TSR) increased gradually with increase of MMT content. For asphalt cement grade (40-50), TSR increased by 3.44 and 11.8% with addition of 2 and 4% of MMT respectively and this increase in TSR returned to decrease at 6% content of MMT, where it increased...
by 6.41% compared with control mixture. For asphalt cement grade (60-70), TSR increased by (4.14, 5.6 and 10)% with addition of (2, 4 and 6)% of MMT respectively. The increase in TSR can be seen in Figures 19 and 20.

![Figure 19. Effect of MMT on the Tensile Strength ratio (TSR) for asphalt cement AC (40-50)](image1)

![Figure 20. Effect of MMT on the Tensile Strength ratio (TSR) for asphalt cement AC (60-70)](image2)

**8.3. Compressive Strength Test Results**

The index of retained strength (IRS.) was used in this test to evaluate the compressive strength of asphalt mixture under the effect of water. According to (SCRB, 2003) the minimum allowed value of IRS is (70%), which is considered as a measure of if the mixture is susceptible to moisture or not. The increase in dry and wet compressive strength and index of retained strength percent for two types of asphalt cement modified by MMT is clear in Figures 21 to 24. For asphalt cement grade (40-50), the dry compressive strength increased by (15.9, 32.3 and 25.6)% with addition of (2, 4 and 6)% of MMT. The modification of asphalt cement with MMT was having a higher effect in wet condition, where the wet compressive strength increased by (29.4, 55.4 and 34.9)% for the same content of MMT. For asphalt cement grade (60-70), the addition of different percentages of MMT by weight of asphalt led to increase the dry and wet compressive strength, where the dry compressive strength increased by (16.1, 28.15 and 26.7)% with addition of (2, 4 and 6)% of MMT respectively. The wet compressive strength continued in increase when (2, 4 and 6)% of MMT were added, where it increased by (25.6, 46.4 and 49.7)% respectively.

For the same reasons recorded in the test of tensile strength ratio (TSR), the addition of MMT made the asphalt mixture have a higher resistance for compressive under the effect of moisture which led to increase the IRS and led to increase moisture resistance.
For asphalt cement grade (40-50), the index of retained strength (ISR) increased by 11.7 and 17.5 for 2 and 4% content of MMT respectively. When the MMT content increased to 6%, the increase in IRS decreased to 7.4% compared with control mixture. For asphalt cement grade (60-70), the index of retained strength (IRS) increased by (8.2, 14.3 and 18)% with addition of (2, 4 and 6)% of MMT respectively. From the results above, we can notice the increase in index of retained strength (IRS) for two types of asphalt cement modified by MMT and this lead to decrease the moisture susceptibility of asphalt mixture, where the moisture resistance increase with increase of IRS.
9. Conclusions

Within the limitations of materials and test program used in this work, the following points are concluded:

- Marshall properties of asphalt mixture were improved with addition of MMT, where the addition of 6% of MMT recorded the highest Marshall stability for two types of asphalt cement, where it was 11.94 and 10.14 kN for asphalt cement AC (40-50) and AC (60-70) respectively compared with control mixture which was 9.45 and 7.93 kN. Also Marshall flow decreased with addition of MMT and the addition of 4% of MMT recorded the highest decrease in Marshall flow for asphalt cement AC (40-50), where it decreased to 2.87 mm compared with control mixture which was 3.47 mm, while the addition of 2% of MMT recorded the highest decrease in Marshall flow for asphalt cement AC (60-70), where it decreased to 3.38 mm compared with control mixture which was 3.64 mm.

- The addition of MMT by weight of asphalt cement lead to decrease moisture susceptibility of asphalt mixture according to the increase in tensile strength ratio (TSR) values. The addition of 4% of MMT gave a highest TSR for asphalt cement grade (40-50) which was 91.23% compared with control mixture which was 81.6%, while the highest value of TSR for asphalt cement grade (60-70) was at 6% content of MMT, where it was 88.41% compared with control mixture which was 80.38%.

- The addition of 4% of MMT also recorded the highest IRS for asphalt cement grade (40-50) which was 85.94% compared with control mixture which was 73.17%, while the highest value of IRS for asphalt cement grade (60-70) was at 6% content of MMT, where it was 84% compared with control mixture which was 71.17%.

10. Funding

This research was funded by Ministry of Higher Education and Scientific Research, University of Baghdad, College of Engineering, Department of Civil Engineering.

11. Conflicts of Interest

The authors declare no conflict of interest.

12. References


Heavy Oil Residues: Application as a Low-Cost Filler in Polymeric Materials

Yulia Yu. Borisova a*, Alsu M. Minzagirova a, Alfina R. Gilmanova a, Mansur F. Galikhanov b, Dmitry N. Borisov a, Makhmut R. Yakubov c

a FRC Kazan Scientific Center of Russian Academy of Sciences, Kazan, Russian Federation.

b Kazan National Research Technological University, Kazan, Russian Federation.

c Arbuzov Institute of Organic and Physical Chemistry, FRC Kazan Scientific Center of Russian Academy of Sciences, Russian Federation.

Received 26 August 2019; Accepted 05 November 2019

Abstract

Deposits of oil sands, bitumen, extra-heavy oil, and heavy oil appear in more than 70 countries all over the world and the fraction of oil recovered gradually increases. High content of poly-condensed high molecular weight oil components (PHMOCs), which may amount up to 50-60% depending on conditions of oil formation, is the main difference of heavy oil and bitumen from conventional oil. PHMOCs can lay the foundation for the preparation of a large number of valuable materials due to their structural manifold and their potential still not discovered to full extent. This work is devoted to the study of the effect of PHMOCs on properties of the composition materials prepared from polyethylene matrix. An «asphalt» – industrial product of deasphalting of tar, as well as asphaltenes and resins isolated from heavy oil, were used as a source of PHMOCs. HDPE and fillers were characterized using MALDI, FTIR, DSC and TGA. For the new composite materials we evaluated the physico-mechanical properties, the thermal decomposition characteristics (by TGA), and the accumulation rate of carbonyl groups in the oxidized polymer (on FTIR). Studies of new composite materials showed that the introduction of filler in an amount of up to 4% in a polyethylene matrix does not lead to a significant change in the physico-mechanical properties, but for a number of parameters they are improved. It also figured out that the addition of PHMOCs to polyethylene makes it unnecessary to stabilize the resulting compositions with stabilizers of thermal oxidative degradation. Results of experimental studies indicate that industrial residue - «asphalt» is a promising filler and low cost of this stock renders it perfect source for the industry of polymer materials.

Keywords: HDPE; Filler; Composition; Heavy Oil; Residue; Asphaltenes; Resins; Thermo-Oxidative Destruction.

1. Introduction

Global oil reserves are 9 to 13 trillion of barrels according to International Energy Agency estimates, among which only 30% is conventional oil, while 70% corresponds to nonconventional oil (30% oil sand and bitumens, 25% extra-heavy oil, and 15% heavy oil). Deposits of oil sands, bitumens, extra-heavy oil, and heavy oil appear in more than 70 countries all over the world and the fraction of oil recovered gradually increases [1]. High content of polycondensed high molecular weight oil components (PHMOCs), which include asphaltenes and resins [2, 3], is the main difference of heavy oil and bitumen from conventional oil. Their total content may amount to 50-60% depending on the chemical nature of oils and conditions of oil formation [4]. Asphaltenes are special among oil components and represent the components with the highest molecular weight and the most complex elemental composition and molecular structure.

*Corresponding author: uborisova@gmail.com

http://dx.doi.org/10.28991/cej-2019-03091432

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
[5-7]. Resins possess lower molecular weight and condensation degree [8], while the alkyl chain length is higher [7]. Although the average molecular weight of resins is lower than that of asphaltenes [9], this is not a significant factor in determining the difference in the behavior of asphaltenes and resins. The main difference is associated with a lower polarity of resins’ molecules [10]. As a result, asphaltenes usually defined as hydrocarbons, which are soluble in aromatic and insoluble in saturated compounds, while resins are insoluble in liquid propane and butane, but soluble in pentane and more highly molecular n-alkanes [11-14].

PHMOCs usually give deposits due to their ability to aggregate and associate [15, 16] and their high content is regarded as an adverse factor for recovery, transport, and upgrading of oil. On the contrary, PHMOCs can lay the foundation for the preparation of a large number of various valuable materials due to their structural manifold and their potential still not discovered to full extent [17-33].

Existing literature data are mainly focused on the study of asphaltenes, on the basis of which many important products are produced (including the compositions with polyethylene, polypropylene, polystyrene, poly(methyl methacrylate), synthetic rubbers, and others), which possess higher performance characteristics than analogous industrial products. However, asphaltenes in pure form are not accessible industrial products. Residual heavy products of oil upgrading containing increased contents of PHMOCs as exemplified by tars, residues of propane–butane deasphalting of tar (asphalt), and others can be a large-scale source of PHMOCs.

Existing literature data mainly focuses on research of asphaltenes. They can be converted into carbon nanomaterials [19-21]; based on them, new composite materials with polyethylene, polypropylene, polystyrene, synthetic rubbers, etc., that have higher performance properties compared with similar industrial products [22-26] are obtained. Modified asphaltenes are widely studied. Asphaltenes modified with silane derivatives as coatings to obtain superhydrophobic nanomaterials to clean oil spills to be applied as an adsorbent in the large-scale removal of petroleum crude oil spills and hydrophobic organic pollutants in marine and aquatic systems [27] and as a novel reinforcing filler in epoxy resin [28]. Asphaltenes modified with styrene-ethylene-butylene-styrene (SEBS) as precursor for isotropic pitch-based carbon fiber [29]; with maleic anhydride - for the synthesis of the polystyrene-asphalene graft copolymer, able to react chemically with paving asphalt components [30]. Asphaltenes modified with acid acquire high ion exchange and adsorption properties [17, 31-33].

However, asphaltenes in pureform are not accessible industrial products, and their modification requires additional costs. Residual heavy products of oil upgrading containing increased contents of PHMOCs as exemplified by tars, residues of solvent deasphalting (SDA) of heavy oil feedstocks can be possible large-scale source of PHMOCs for creating new polymer compositions. SDA is one of the most interesting technologies for the processing of heavy residues at modern oil refineries that requires little investment [34-36]. During the SDA process, in the presence of low molecular weight alkanes or other precipitants, in regard to which asphaltenes are lyophobic, these precipitators coagulate and entrain high molecular weight asphaltene-resinous substances in the form of solvate layers. When deasphalting is aimed at separating deasphalted oil (DAO), SDA is carried out exclusively using propane, which allows selecting the optimal amount of oil fractions of quite good quality and carrying out the process at moderate temperatures and pressures. If the target product is the residue, (PHMOCs concentrate) - «asphalt», then heavier alkanes are usually used [37].

This work is devoted to the study of the effect of oil fillers on physicomechanical and other properties of the composition materials prepared from polyethylene matrix. The results obtained for the compositions on industrial residue of propane–butane deasphalting of tar, as well as asphaltenes and resins isolated from heavy oil are compared. Particular attention was devoted to the evaluation of the performance of the mentioned fillers as stabilizers of thermo-oxidative destruction.

2. Experimental Section
2.1. Materials

HDPE produced in OAO Kazan’orgsintez were used, namely, PE1 (counterpart of PE100) corresponding to the polyethylene brand for the production of pipes and PE2 brand devoted to the fabrication of household and business products. Asphalt is an industrial residue of propane–butane deasphalting of tar produced by Novoufimsk oil refinery, as well as asphaltenes and resins, which were isolated from heavy oil of Zyuzeevo deposit (Tatarstan, Russia). Composition and properties of the oil objects are given in Table 1 and elemental composition of polyethylenes and the oil objects (fillers) are given in Table 2.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Density (at 20°C) (g/cm³)</th>
<th>Viscosity (at 20°C) (mm²/s)</th>
<th>Asphaltenes (wt %)</th>
<th>Resins (wt %)</th>
<th>Saturated and aromatics hydrocarbons (wt %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>asphalt</td>
<td>1.0762</td>
<td>-</td>
<td>13.7</td>
<td>40.7</td>
<td>45.6</td>
</tr>
<tr>
<td>oil</td>
<td>0.9250</td>
<td>472</td>
<td>6.9</td>
<td>26.9</td>
<td>66.2</td>
</tr>
</tbody>
</table>

Table 1. Physicochemical properties and component composition of petroleum samples
2.2. Methods and Equipment

Asphaltenes were precipitated from the heavy oil by 20-fold volume excess of n-hexane. After 24 h, the obtained precipitate was filtered and washed with boiling n-hexane in a Soxhlet apparatus up to decolorization of flowing solvent to remove as much maltenes as possible. The solvent from the maltenes solution was removed up to a constant weight using a rotary evaporator (60 mm Hg/30°C). Maltenes were separated by elution column chromatography on silica gel [38]. n-Hexane were used to elute saturated and aromatics hydrocarbons, the blend of isopropanol/benzene (1:1, v/v) - for the extraction of resins.

The polymer was mixed with filler on a Brabender mixer for 5 min at 190°C. PE specimens with asphalt, asphaltenes, resins, and Anox® 20 Powder were prepared in Figure 1. The plates that are 1 mm in thickness were prepared by pressing at the temperature of 170±5°C and exposed to pressure for 5 min.

Figure 1. Flowchart of polyethylene compositions’ research

Tensile strength (TS), Elongation at break (ε), Elastic Modulus (E), and Yield strength (σ) of the compositions were determined according to ISO 527-2:2012 on an Inspect mini tensile machine. The melt flow rate (MFR) of polyethylene and its compositions was determined on an IIRT-5М plastometer at 190°C and the load of 49 N (5 kgs). Density was determined according to ASTM D792 – 13.

Elemental composition was determined using a CHNS-O Euro EA3028-HT-OM analyzer (EuroVector). Matrix-assisted laser desorption/ionization (MALDI) mass spectra were obtained by Ultra Flex III TOF/TOF mass-spectrometer (Bruker Daltonik GmbH, Bremen, Germany) in a linear mode with Nd:YAG laser (λ = 266 nm). Spectra were obtained with 25 kV of accelerating voltage and 30 ns of acceleration delay. The data were processed by using Flex Analysis 3.0 software (Bruker Daltonik GmbH, Bremen, Germany). 1, 8, 9-Trihydroxyanthracene was used as a matrix.

Infrared (IR) spectra of the compounds were recorded in the range of 4000–400 cm⁻¹ on a Tensor-27 IR-Fourier spectrometer (Bruker) with the optical resolution of 4 cm⁻¹. Polyethylene specimens were prepared in the form of 1-mm plates after preliminary melting. The spectra were processed and analyzed using OPUS software (Bruker Optik GmbH).

Thermal analysis was carried out on a Q-1500D derivatograph of MOM Company (Hungary) in the temperature range of 20-1000 °C at the heating rate of oven of 10°C/min. Air atmosphere in the oven is stationary. Alumina was used as inert substance. The shot of specimen was 60 mg. Melting point Tm (the temperature of minimum at corresponding section of DTA curve), stages of thermo-oxidative destruction (I-IV; the character of heat effect is also

---

<table>
<thead>
<tr>
<th>Sample</th>
<th>C</th>
<th>H</th>
<th>N</th>
<th>S</th>
<th>H/C</th>
</tr>
</thead>
<tbody>
<tr>
<td>PE1</td>
<td>84.70</td>
<td>15.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PE2</td>
<td>80.52</td>
<td>14.96</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>asphalt</td>
<td>83.79</td>
<td>9.53</td>
<td>0.24</td>
<td>4.87</td>
<td>1.36</td>
</tr>
<tr>
<td>asphaltenes</td>
<td>79.72</td>
<td>8.28</td>
<td>1.62</td>
<td>10.26</td>
<td>1.25</td>
</tr>
<tr>
<td>resins</td>
<td>75.96</td>
<td>8.92</td>
<td>1.25</td>
<td>9.39</td>
<td>1.41</td>
</tr>
</tbody>
</table>
indicated), and the temperature corresponding to the maximum weight loss Tmax were determined for each polyethylene specimen.

DSC curves of PE were recorded on a C80 calorimeter of SETARAM Company. Temperature scanning of a 50-mg shot was carried out in the range from 20 to 200°C at the rate of 1° min⁻¹. The calorimeter was calibrated by Joules effect and melting point In; the temperature measurement error was less than 0.1°C. The melting point of the crystal phase corresponded to the temperature of minimum on the heat effect of melting. In Table 3, melting points Tm and melting enthalpy (ΔHm) are given. The degree of crystallinity was calculated by division of the melting enthalpy of PE specimen, ΔHm, by the melting enthalpy of PE with 100% crystallinity. It is suggested that the melting enthalpy of 100% crystalline PE is 293 J/g [24].

Performance of the fillers as stabilizers of thermo-oxidative destruction was evaluated according to TGA data (Tm and the temperature corresponding to 5, 10, and 50% weight loss of the specimen) and the accumulation rate of carbonyl groups in oxidized polymer using FTIR. The plate of the composition specimen with the size of 10 mm×10 mm×1 mm on degreased glass plate was transferred to a drying oven for thermal oxidation in the air. Thermal oxidation was carried out at 190°C (precision of temperature maintenance is ±1°C) within 20 h. IR spectra were recorded before thermal oxidation after 5, 10, 15, and 20 h on a Bruker Tensor 27 FT-IR spectrometer equipped with Hyperion IR microscope (Bruker) in the range of 7500-560 cm⁻¹. The absorption band at n =1720 cm⁻¹ corresponding to stretching vibrations of carbonyl groups of ketone and aldehyde type was used as analytical band for quantitative analysis. The bands at n =1366-1368 cm⁻¹ and 1305-1309 cm⁻¹, which are related to bending (wagging and twisting, respectively) vibrations of CH₂ and CH₃-groups, were chosen as internal standard. The oxidation state of polymer was evaluated according to the ratio of relative-to-integral intensities at n =1720 cm⁻¹ and n =1366-1368 cm⁻¹ or 1305-1309 cm⁻¹.

3. Results and Discussion

3.1. Characterization of PE and Fillers

Determination of the intensity ratio of spectral bands I₁₃₆₈/I₁₄₇₂ corresponding to bending vibrations of CH₂ (~1472 cm⁻¹) and CH₃ groups (~1368 cm⁻¹) is one of the variants to evaluate the quality of polyethylenes. That is, the lower the ratio, the less the number of CH₃ groups and the more homogeneous polyethylene is. In order to determine the crystal structure of polyethylene, one can use the doublet of rocking vibrations at 720 and 730 cm⁻¹, which is associated with the Davydov splitting in orthorhombic cell (the cell contains two fragments of C₂H₄). There is only one intense absorption band at 720 cm⁻¹ in triclinic crystals of polyethylene (one cell contains one fragment of C₂H₄) and in the amorphous phase [39]. In addition, these bands become narrower and better resolved with an increase in the degree of crystallinity. IR spectra of the PE specimens in Figure 2 are quite similar in the range of 720-730 cm⁻¹ and 1300-1500 cm⁻¹, which indicates identical quality of polyethylene and similar degree of crystallinity.
According to DSC data in Table 3, PE2 specimen possesses higher melting point $T_m$ (133.7 °C) and higher degree of crystallinity $X_{cryst}$ (68.1%), which is intrinsic for linear LDP with a small quantity of short side chains. High melting point (132.9 °C), as well as the medium degree of crystallinity (nearly 58%) of PE1 specimen indicates a linear character with a large number of short side chains, which restrict crystallization processes.

Table 3. DSC data of PE

<table>
<thead>
<tr>
<th>Sample</th>
<th>$T_m$ °C</th>
<th>$\Delta H_m$ J/g</th>
<th>$X_{cryst}$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>PE1</td>
<td>132.9</td>
<td>171.1</td>
<td>58.4</td>
</tr>
<tr>
<td>PE2</td>
<td>133.7</td>
<td>199.6</td>
<td>68.1</td>
</tr>
</tbody>
</table>

Several stages of destruction process can be differentiated on DTA curves of PE specimens in Table 4. Endothermic peak of melting is recorded in the temperature range of 131-133 °C, which is intrinsic for LDP or HDPE. Four stages of thermo-oxidative destruction are recorded on DTA curve with a subsequent heating. I stage of decomposition of the material, which is related to its low-temperature oxidation, is caused by the elimination of low-molecular fraction in polymer, as well as the presence of weak bonds, whose cleavage initiates destruction of carbon–carbon fragments to give free radicals. Oxidizing process becomes more complicated with further heating of the specimens and exothermic (II stage) and endothermic (III stage consisting of three endotherms) effects are recorded on DTA curve. These transformations are related to deep processes of decomposition of the main chain and evolution of a wide variety of organic saturated and unsaturated volatile monomers. Complete deposition of PE occurs at stage IV (almost by 100%) with the appearance of exothermic effect.

Table 4. TGA data of PE

<table>
<thead>
<tr>
<th>Sample</th>
<th>$T_m$ °C</th>
<th>I, exo</th>
<th>II, exo</th>
<th>III, endo</th>
<th>IV, exo</th>
<th>$T_{max}$ °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>PE1</td>
<td>132</td>
<td>186-309*</td>
<td>309-421</td>
<td>421-522</td>
<td>517-581</td>
<td>485</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.1</td>
<td>14.4</td>
<td>76.3</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>PE2</td>
<td>133</td>
<td>188-321</td>
<td>321-428</td>
<td>428-520</td>
<td>520-579</td>
<td>490</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.1</td>
<td>14.0</td>
<td>74.3</td>
<td>2.2</td>
<td></td>
</tr>
</tbody>
</table>

* In the numerators– temperature range in °C, in the denominator - weight loss in this temperature range (%).

Comparison of thermal analysis data of PE specimens indicates that the PE specimens are characterized by almost identical thermo-oxidative destruction stages; PE2 specimen is characterized by higher content of low-molecular fraction, as well as the presence of weak bonds.

In turn, temperature zone of thermo-oxidative degradation of oil fillers in Table 5 can be divided into three range, namely, 20–400, 400–500, and 500–770 °C. At the first stage (temperature range is 20–400 °C), there is evaporation of hydrocarbons, oxidation of methylene and methyl groups, and decomposition of carboxylic groups. At the second
stage (temperature range is 400–500 °C), there is subsequent decomposition of the carboxyl–carbonyl groups, cleavage of peripheral groups from aromatic structures, and probably the condensation [40] of aromatic rings. At third stage (temperature range is 500–770 °C), there is a combustion of condensed naphthene-aromatic structure.

### Table 5. TGA data of petroleum fillers

<table>
<thead>
<tr>
<th>Sample</th>
<th>Mass loss/mass% in temperature interval</th>
<th>A</th>
<th>F</th>
<th>P</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Δm1 20–400°C</td>
<td>Δm2 400–500°C</td>
<td>Δm3 500–700°C</td>
<td></td>
</tr>
<tr>
<td>asphalt</td>
<td>18.1</td>
<td>42.8</td>
<td>39.1</td>
<td>1.6</td>
</tr>
<tr>
<td>asphaltenes</td>
<td>8.4</td>
<td>42.4</td>
<td>49.2</td>
<td>1.0</td>
</tr>
<tr>
<td>resins</td>
<td>20.7</td>
<td>47.3</td>
<td>32.0</td>
<td>2.1</td>
</tr>
</tbody>
</table>

To characterize oil fillers, we used the parameter $A = (\Delta m1+\Delta m2)/\Delta m3$, which characterizes the ratio of sum of paraffin–cycloparaffin hydrocarbons and alkyl substituents to naphthene-aromatic structures, $F = \Delta m1/(\Delta m2+\Delta m3)$, which reflects the mass ratio of light and medium components to heavy, and $P = \Delta m2/\Delta m3$, which reflects the mass fraction of peripheral substituents in cyclic structures [41].

Analysis of TGA data showed that asphaltenes are characterized by the highest thermal stability and the maximum weight loss occurs at the temperature higher than 400 °C. On average, asphaltene molecule contains less aliphatic structures, as well as lateral substituents in condensed naphthene-aromatic structures. Resins contain a larger number of alkyl substituents (A), while the weight fraction of peripheral substituents (P) in cyclic structure is almost two times as large as that in asphaltenes. The parameters in the case of asphalt containing asphaltenes and resins, as well as saturated and aromatic hydrocarbons in Table 1, are intermediate in Table 5.

Because asphaltenes and resins are formed by a continuous series of various hybrid high-molecular compounds of oil, only average molecular weight of these oil components can be considered. The average molecular weight is 1700 a.m.u. in the case of asphaltenes and 600 a.m.u. in the case of resins according to MALDI mass spectra. In the case of the resins isolated from asphalt in Table 1, the average molecular weight is highest, 819 a.m.u., while in the case of asphaltenes it is 1700 a.m.u.

It is mentioned in [22] that the structures, in which the hydrogen-to-carbon ratio (H/C) is less than 1.40–1.30 according to elemental analysis data, are preferred as fillers of composition materials. In our case in Table 2, the H/C ratio in asphalt is 1.36, while it is 1.25 in asphaltenes and 1.41 in resins.

#### 3.2. Physicomechanical Properties

It was indicated [23, 24] that most physicomechanical properties worsen at high degree of filling of the polymer matrix with asphaltenes; therefore, we prepared the compositions with 4 wt % of asphalt and 4 wt % of asphaltenes. To determine the effect of resins on the characteristics of the compositions, we used additives at small quantities, namely, 0.1 and 0.2%, and also considered their effect in asphalt (1.6% of resins calculated from Table 1). We also presented some data for HDPE trademarks*, which already contain industrial additives (including 0.2 wt % of Anox® 20 Powder stabilizer). The specimens prepared from PE with 0.2 wt % of Anox and 0.2 wt % Anox with 4 wt % of asphaltenes were also compared.

The studies showed that the change of physicomechanical characteristics of the compositions does not have a clear dependence on their composition in Table 6.

### Table 6. Physicomechanical properties of polyethylenes and their compositions

<table>
<thead>
<tr>
<th>Samples (% wt)</th>
<th>TS, MPa</th>
<th>$\varepsilon_0$ %</th>
<th>$E_0$, MPa</th>
<th>$\sigma_0$, MPa</th>
<th>MFR, g/10min</th>
<th>$\rho_0$, g/sm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>HDPE1*</td>
<td>&gt;21*</td>
<td>&gt; 500*</td>
<td>not regulated</td>
<td>not regulated</td>
<td>&gt;0.1 63.2**</td>
<td>0.96**</td>
</tr>
<tr>
<td>PE1</td>
<td>35.1</td>
<td>1068</td>
<td>416</td>
<td>26</td>
<td>0.049</td>
<td>0.96</td>
</tr>
<tr>
<td>PE1 + Anox (0.2)</td>
<td>30.7</td>
<td>895</td>
<td>407</td>
<td>27</td>
<td>0.044</td>
<td>0.96</td>
</tr>
<tr>
<td>PE1 + asphalt (4)</td>
<td>29.7</td>
<td>850</td>
<td>580</td>
<td>29.7</td>
<td>0.16</td>
<td>0.963</td>
</tr>
<tr>
<td>PE1 + asphaltenes (4)</td>
<td>26.8</td>
<td>663</td>
<td>506</td>
<td>26.7</td>
<td>0.16</td>
<td>0.963</td>
</tr>
<tr>
<td>PE1 + asphaltenes (4) + Anox (0.2)</td>
<td>25.5</td>
<td>850</td>
<td>434</td>
<td>25.4</td>
<td>0.18</td>
<td>0.963</td>
</tr>
<tr>
<td>PE1 + asphaltenes (4) + resins (0.2)</td>
<td>27.8</td>
<td>770</td>
<td>541</td>
<td>28.4</td>
<td>0.15</td>
<td>0.963</td>
</tr>
<tr>
<td>PE1 + resins (0.2)</td>
<td>28</td>
<td>760</td>
<td>459</td>
<td>29.1</td>
<td>0.12</td>
<td>0.956</td>
</tr>
<tr>
<td>PE1 + resins (0.1)</td>
<td>29.1</td>
<td>770</td>
<td>507</td>
<td>29.6</td>
<td>0.11</td>
<td>0.958</td>
</tr>
<tr>
<td>HDPE2*</td>
<td>&gt;30*</td>
<td>not regulated</td>
<td>not regulated</td>
<td>&gt;26*</td>
<td>&gt;2.3**</td>
<td>0.957-0.963**</td>
</tr>
</tbody>
</table>

2559
One example is that an increase in the elastic modulus (E) of polymers with the addition of disperse and not only disperse filler is not surprising and is a universal feature of all the composition systems. This occurs due to the fact that the elastic modulus of solid and unrelated filler particles is higher than the elastic modulus of polyethylene. It is evident that replacement of the fraction of bulk polymer with other particles decreases the strain capacity of the composition and increases its resistance to deformation.

An increase in the elastic modulus is also often associated with the presence of adsorbed polymer macromolecules on the surface of filler particles. One part of the macromolecule is adsorbed on surface and immobilized. This immobilization is transferred to a particular distance along the macromolecule decreasing its flexibility, which affects strain characteristics of the polymer composition. Consequently, a slight increase in the rigidity of polymer is observed with an increase in the filler content [23, 24].

It can be suggested that, in the case of PE1, interaction of polymer and filler is weaker at the interface, while in the case of PE2 compositions the presence of resins gives a more drastic increase in the elastic modulus E, in particular, in the case of asphalt in Figure 3.

![Image](image_url)  
**Figure 3. Effect of the composition of polyethylene compositions on elastic modulus (E)**

Deformation of solid under external mechanical loads is caused not only by the change of the conformations of macromolecules, but also displacement of macromolecules relative to each other (flow). An increase in the tensile yield strength $\sigma$ is observed almost for all PE compositions. It is clear from Figure 4 that the presence of resins, both individually and mixed with other components in the composition, increases the ductility of the polymer material. It is
known from the literature [17, 42] that petroleum resins are used to improve the ductility and adhesion of bitumen, as well as to reduce internal friction during processing (mixing, molding) of industrial rubber goods. The observed effect can also be associated with the structural features of petroleum resins’ molecules, which have long alkyl chains and have surface-active properties [7, 10].

In the case of melt flow rate (MFR) parameter, there is a 4–6-fold increase as compared to the initial PE in Table 6. MFR is widely used in production and characterizes the viscosity of melts, average molecular mass, and the degree of branching of thermoplastics [43–46]. In this case, probably, the formation of many CH-π hydrogen bonds [47] initially occurs on both sides of a large number of polycondensed aromatic systems of oil fillers. This leads to a change in the spatial globular structure of polyethylene with the orientation of the macromolecules along the filler macromolecules, and, as a consequence, to an increase in the mechanical strength of the composite. When the temperature rises to 190 °C, during determination of the MFR, these intermolecular hydrogen bonds are destroyed, but due to the viscous-flow state of polyethylene, its spatial structure does not have time to return to its original state, which in turn leads to an increase in its fluidity.

Figures 5 and 6 demonstrate the tensile strength and elongation at break of the PE compositions. The best result is demonstrated by polyethylene compositions with asphalt, with resins and asphaltenes, with resins whereas, compositions with only asphaltenes show lower values. Probably, under given conditions for mixing, asphaltenes are insufficiently well distributed in the polymer material, forming small agglomerates [23–25], which are a source of stresses. Observed increase in strength (especially for PE2) can be explained by the plasticizing effect of resins, which also contribute to a better distribution of asphaltenes when used together in a polymer composition.
3.3. Thermo-oxidative Degradation

The effect of resin-asphaltene substances on the performance characteristics of liquid fuels, oils, and polymers were carried out in [48-50]. Analysis of the results of inhibition effect of oil stabilizers (OSs) in the model oxidation reaction of cumene showed that they include nearly 10% of high-performance stabilizers [48]. It was determined that the most effective fraction of OS contains the largest fraction of high-condensed aromatic structures with heteroatomic groups.

Investigation of thermal destruction of polyethylene, polypropylene, polystyrene, poly(methyl methacrylate), and synthetic rubbers confirmed high performance of asphaltenes as stabilizers [23-25,48]. In this case, the price of OSs is
several orders of magnitude less than that of synthetic stabilizers as exemplified by Anox and other phenolic antioxidants.

The performance of the oil fillers as stabilizers of thermo-oxidative destruction was analyzed according to the accumulation rate of carbonyl groups in the oxidized polymer using IR spectrophotometry data, as well as TGA method.

In Figures 7 to 10, the dependences of the oxidation level of the PE composition calculated according to the ratio of relative intensities of the compositions at 1720 cm\(^{-1}\) (C-O bond of ketone and aldehyde types) to 1366-1368 cm\(^{-1}\) (CH3-groups) and 1305 cm\(^{-1}\) (CH2-groups) in IR spectra vs. time of thermal oxidation of the compositions are given.

**Figure 7.** Dependence of the oxidation level \(f\) of PE1 compositions (I\(_{1720}/I_{1366-1368}\)) on the time of thermal oxidation

**Figure 8.** Dependence of the oxidation level \(f\) of PE1 compositions (I\(_{1720}/I_{1305}\)) on the time of thermal oxidation
As a result, it was determined that resins do not stabilize polymer. On the contrary, the compositions containing asphaltenes, asphalt, and asphaltenes with resins demonstrate high thermal stability, which is comparable to that of the compositions of initial polyethylenes with Anox and industrial trademarks HDPE® containing stabilizers. It has been
established that inhibiting ability of thermo-oxidative processes is associated with the number of paramagnetic centers in the stabilizer molecule [17]. Due to the presence of paramagnetic centers, resins and asphaltenes are recommended as polymer stabilizers, while the number of paramagnetic centers and the rate of inhibition constant of the latter are 1-2 orders of magnitude higher.

The form of the change of the oxidation level (f) of all compositions with OSs on time is nonlinear. The first peak of f (at the times of thermo-oxidative destruction under study) appears at 5 h; then, it decreases in most cases and, after that, the second peak appears at 15 h; finally, this value decreases in all compositions. The specimens stabilized with Anox, on the contrary, firstly demonstrated the maximum protective effect; however, the oxidation level of the specimens increased with an increase in the ageing period of the polymer.

It is considered that phenolic antioxidants act as H donors and prevent the formation of hydroperoxide sequences and cleavage of primary bonds assuming that their concentration in the polymer remains higher than critical value. However, consumption of stabilizer at long-term ageing is intrinsic for them [51]. On the contrary, OSs are multifunctional, which is caused by the presence of condensed naphthene–arene structures, as well as heterofunctional groups [48, 50]. A long-term slowdown of radical processes, which is caused by the activation of inhibiting centers with oxygen and temperature, and formation of secondary inhibitors are intrinsic for them [50].

In analogy, high thermal stability according to TGA data is observed in all compositions of PE with fillers, except for the compositions with resins in Table 7. Though asphaltenes are considered best OSs [17, 48], asphalt exhibits comparable protective properties at the same order (T_{5\%}, T_{10\%}, and T_{50\%}).

<table>
<thead>
<tr>
<th>Sample (wt.%)</th>
<th>T_{\text{m}, \circ C}</th>
<th>T_{5%}</th>
<th>T_{10%}</th>
<th>T_{50%}</th>
</tr>
</thead>
<tbody>
<tr>
<td>HDPE1’</td>
<td>127</td>
<td>355</td>
<td>385</td>
<td>479</td>
</tr>
<tr>
<td>PE1</td>
<td>132</td>
<td>266</td>
<td>344</td>
<td>466</td>
</tr>
<tr>
<td>PE1 + Anox (0.2)</td>
<td>134</td>
<td>359</td>
<td>407</td>
<td>486</td>
</tr>
<tr>
<td>PE1 + asphaltenes (4)</td>
<td>131</td>
<td>315</td>
<td>373</td>
<td>469</td>
</tr>
<tr>
<td>PE1 + asphalt (4)</td>
<td>128</td>
<td>331</td>
<td>369</td>
<td>469</td>
</tr>
<tr>
<td>PE1 + asphaltenes (4) + Anox (0.2)</td>
<td>129</td>
<td>322</td>
<td>371</td>
<td>486</td>
</tr>
<tr>
<td>PE1 + asphaltenes (4) + resins (0.2)</td>
<td>127</td>
<td>310</td>
<td>348</td>
<td>470</td>
</tr>
<tr>
<td>PE1 + resins (0.1)</td>
<td>127</td>
<td>280</td>
<td>340</td>
<td>470</td>
</tr>
<tr>
<td>PE1 + resins (0.2)</td>
<td>125</td>
<td>309</td>
<td>352</td>
<td>464</td>
</tr>
<tr>
<td>HDPE2’</td>
<td>138</td>
<td>274</td>
<td>340</td>
<td>472</td>
</tr>
<tr>
<td>PE2</td>
<td>133</td>
<td>321</td>
<td>376</td>
<td>462</td>
</tr>
<tr>
<td>PE2 + Anox (0.2)</td>
<td>139</td>
<td>367</td>
<td>414</td>
<td>488</td>
</tr>
<tr>
<td>PE2 + asphaltenes (4)</td>
<td>137.5</td>
<td>301</td>
<td>370</td>
<td>482</td>
</tr>
<tr>
<td>PE1 + asphalt (4)</td>
<td>130</td>
<td>300</td>
<td>391</td>
<td>478</td>
</tr>
<tr>
<td>PE2 + asphaltenes (4) + Anox (0.2)</td>
<td>138</td>
<td>362</td>
<td>410</td>
<td>486</td>
</tr>
<tr>
<td>PE2 + asphaltenes (4) + resins (0.2)</td>
<td>135</td>
<td>327</td>
<td>376</td>
<td>464.5</td>
</tr>
<tr>
<td>PE1 + resins (0.1)</td>
<td>133</td>
<td>316</td>
<td>377</td>
<td>469.5</td>
</tr>
<tr>
<td>PE1 + resins (0.2)</td>
<td>130</td>
<td>300</td>
<td>356</td>
<td>466</td>
</tr>
</tbody>
</table>

* Manufacturing requirements of HDPE trademark.

4. Conclusions

- Studies of the HDPE and PHMOC compositions have shown that introduction of up to 4% of filler to the polyethylene matrix does not significantly change physicomechanical properties, while some parameters have shown an increase.

- It has been determined that addition of asphalt and asphaltenes both individually and in the mixture with resins to polyethylenes does not require stabilization of the composition using stabilizers of thermo-oxidative destruction.

- Comparison of the oil fillers has demonstrated that resins cannot be employed as individual additive, while asphaltenes possess some drawbacks, which are related to nonhomogeneous distribution in the polymer matrix. At the same time, resins provide higher dispersion of asphaltenes in the polymer matrix and improvement of viscoelastic characteristics of the composition and asphaltenes results in the stability of the composition to thermal destruction. Employment of the oil fillers containing both asphaltenes and resins is optimal.
• Results of experimental studies indicate that industrial residue of propane–butane deasphalting of tar, more specifically, asphalt containing asphaltenes and resins, is a promising filler and low cost of this stock renders it perfect source for the industry of polymer materials.

5. Funding

Financial support from the government assignment for FRC Kazan Scientific Center of RAS.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Behavior of Reinforced Concrete Beams with effect of Stiffened Plates

Ali Sabah Al Amli a, b,*, Laith Shakir c, Ali Abdulredha d, Nadhir Al-Ansari b

a Al-Mustansiriya University, Palestine Street, 10052 Baghdad, Iraq.
b Lulea University of Technology, 971 87 Lulea, Sweden.
c University of Karbala, 56001 Karbala, Iraq.
d University of Warith AL-Anbiya’a, 56001 Kerbala, Iraq.

Received 31 May 2019; Accepted 19 September 2019

Abstract

This study presents experimental work including an investigation conducted on five simply supported reinforced concrete beams under pure torsion. First beam without strengthening as a control beam. The other four beams were strengthened externally by bolted thin steel plates. For this test the load was applied gradually. The torque was increased gradually up to failure of the beam. The variables were the thickness and height of the steel plate that was externally connected to both sides of the rectangular reinforced concrete beam. The test results for the beams discussed are based on torque-twist behavior. The experimental results show that the attachment of thin steel plates by mechanical means to beams provides a considerable improvement in the torsional behavior of the reinforced concrete beams. Comparable to the reference beam, the maximum increase in the cracking and the ultimate torque of the composite beam was recorded for the reinforced concrete beam that strengthen by steel plate of 150 mm height, 2 mm thickness and 50 mm spacing between shear connectors (B1). The results revealed that the cracking torque, ultimate torque, global stiffness of beam and beam ductility for all composite beams increase with the increase of the plate's thickness, plate's height.

Keywords: Torque; Reinforced Concrete; Steel Plate; Strengthening, Beams.

1. Introduction

The torsion effects were omitted from the design of reinforced concrete structures for many years. In 1958 ACI committee 438 had made recommendations for suitable torsion design requirement [1]. As a result of these efforts, the ACI-318 Building Code [2] was inserting a new form of torsion design criteria at 1971 for the first time.

Before cracking of reinforced concrete members under torsional moment, there was perceivable effect on the stiffness from reinforcement. As the same, an additional strength beyond the plain concrete capacity can be gain from the longitudinal or transverse reinforcement [3].

First cracking torque is commonly increased, when the longitudinal and the transverse steel are combined. According to the value and location of the reinforcement, a considerable increase in strength and a large amount of plastic torque are possible in spite of stiffness reducing after cracking moment [4].

Fang and Shiau [5] presented experimental results of torsional behavior for normal and high strength concrete beams (NSC and HSC, respectively). Different values of reinforcement were used, subjected to pure torsion. The experimental results demonstrated that the high strength concrete beams owned higher cracked stiffness and torsional strength than
the normal concrete beams fabricated with the same amount of reinforcement. After the ultimate strength, the high strength beams showed relatively a high decay in strength than the normal strength beams high reinforcement design.

The torsional capacity of peak point on rectangular beams section with both spiral and tied reinforcements in the torsion direction and its anti-direction was investigated by Barghlame and Lotfollahi-Yaghn [6]. The tests showed that the prismatic beam with spiral links has lower torsion capacity than the tied ones.

Ghobarah et al. [7] performed an experimental test on using fiber reinforced polymer (FRP) to enhance the torsion strength of reinforced concrete beams. It was observed that the using wrapping configuration were very effective especially for fully wrapped beams which give behavior better than the beams reinforced with stirrups. It was shown from all the tests, the applicability of using FRP to improve the torsional resistance of reinforced concrete beams.

Panchacharam and Belarbi [8] concluded that, the cracking and the ultimate torsional capacity were significantly improved by strengthening the reinforced concrete beams with externally GFRP sheets.

A nine reinforced concrete beams strengthened with an external GFRP fabrics were executed under the torsional moments by Tudu [9]. Different patterns with different kinds of GFRP fabrics were distributed on eight beams, while the ninth one represents control beam. The results deduced that using GFRP fabrics as external stiffening can enhance the torsional capacity of beams greatly, also the results showed that the best GFRP fabrics patterns is the full wrap. As well as adding GFRP in 45° with longitudinal axis of beams shows high strength than the applied ones in 90° with beam axis.

In this study only square beams subjected to pure torsional moment are investigated. Thin steel plates attached to a reinforced concrete beam by means of mechanical connectors. The function of these connectors is to transfer the forces between the two components.

The aim of the current work is to investigate the different thicknesses and heights of steel plate on the behavior of reinforced concrete beams strengthened with steel plates under pure torsion. The cracking torque, ultimate torque, mode of failures, and angle of twist of the beams are discussed through this study. Many researches were studied the behavior of beams with many sections, materials, reinforcement and repairing with the Torque effect [10-14].

The experimental procedure consists of testing five composite reinforced concrete-beam specimens subjected to pure torsion. One of which used as the control beam without plate (BN), the other four beams with steel plates of two different thicknesses (1.5 mm and 2 mm) were subjected to monotonic loading. All the beams tested have the same dimensions: 100 mm width, 200 mm depth and 1500 mm length. The compressive cubic strength of all tested beams was about (30MPa) at age of (28) days. The steel reinforcements ratio used in all the composite concrete-beam sections were similar (ρ= 0.91%), while the spacing of closed stirrups is 100mm c/c. The variables studied through the procedure test were the thickness and height of the externally attached steel plate: B1 strengthened externally with steel plate of 2 mm thickness and 150 mm height, B2 strengthened externally with steel plate of 1.5 mm thickness and 150 mm height, B3 strengthened externally with steel plate of 2 mm thickness and 100 mm height, and B4 strengthened externally with steel plate of 1.5 mm thickness and 100 mm height.

Two deformed steel bars diameters are used through this investigation, the first one is (10mm) which are used for the main reinforcement and the second steel bars is (8mm) which are used for stirrups as shown in Figure 1. From the results, the adopted steel bars conformed to (ASTM A615-86) [15] as shown in Table 1. While two different steel plates thicknesses of (1.5, 2) mm were used to strengthen the composite reinforced concrete beams, which the properties are shown in Table 2, and the material properties [20]. Figure 2 shows the steel plates arrangement for beams specimens. Many previous researches studied the effect of torque [16-25]. This study is very important to increase the torque capacity to the beams. Also, this method can used to repairing the beams or increase the stiffness of them. When the capacity increased, the structure with these beams is suitable in design and don’t need rehabilitation in the future; thus, good for the cost.

<table>
<thead>
<tr>
<th>Diameter Bar (mm)</th>
<th>Measured Diameter (mm)</th>
<th>Yield Stress * (N/mm²)</th>
<th>Ultimate Strength (N/mm²)</th>
<th>Elongation %</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>8</td>
<td>496</td>
<td>612.9</td>
<td>17</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>521</td>
<td>615.7</td>
<td>19</td>
</tr>
</tbody>
</table>

* Each value is an average of three specimens.
Figure 1. Details of Beams Reinforcement: (A) cross section of steel reinforcement; (B) reinforcement of the concrete beam

Table 2. Steel Plates Specifications

<table>
<thead>
<tr>
<th>Thickness of steel specimens (mm)</th>
<th>Yield Stress (N/mm²)</th>
<th>Ultimate Strength (N/mm²)</th>
<th>Elongation* (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>326.3</td>
<td>394</td>
<td>17.6</td>
</tr>
<tr>
<td>2</td>
<td>386.3</td>
<td>426.6</td>
<td>15.2</td>
</tr>
</tbody>
</table>

* Each value is an average of three specimens.

Figure 2. Steel plates arrangement for beams specimens: (A) beam B1; (B) beam B2; (C) beam B3; (D) beam B4

2. The Procedure of Test

The procedure of tests (The Methodology) is expressed as following:

- All beams were tested under monotonically increasing torque up to failure. For this test the load was applied gradually.
- The torque was increased gradually up to failure of the beam; Figure 3 shows the test setup.
- The Universal Testing Machine model (8551 MFL system) with a maximum capacity of 300 tons was used to apply the torsional moment on the tested beams with a structural arrangement shown in the Figure 4.
- The proposed arrangement of test was conducted by Zararis and Penelis [26].
• The rigid clamping loading frame on each end of the tested beam used in this study.

• The torque is satisfied by the arms with separated faces to connect them over the sample with four large bolts for each arm, while a steel girder of 250 mm depth and 2500 mm length was used to transmit the loads from the center of the universal machine to the two arms (pure torsion) as shown in Figure 4.

• Two dial gauges tied to the bottom fiber of the end of the beam at a point 35 mm from the center of the longitudinal axis were used to estimate the angle of twist, Figure 5. Two dial gauges, the first one on the right and the other on the left, recorded the uplift and downward values to estimate the twist angle.
3. Experimental Results

3.1. Effect of Steel Plate Height on Torsional Behavior

To understand the effect of steel plate height on the reinforced concrete beams strengthened with steel plates, two heights (100 and 150 mm) were used in the current work. Tables 3 and 4 describe the general test results for the reinforced beams, whereas the effects of plate height on the torsional resistance of the tested beams are covered by Figures 7 to 10.

Test results indicate that the steel plate height made a good contribution to the crack and ultimate torque resistance, where both the cracking and ultimate torque increased as the height of the steel plate increased. The tested beams attached to the steel plate of 100 mm in height exhibited an increase in cracking and ultimate torques (from 124% to 200% and 151% to 222% for steel plate thickness 1.5 and 2 mm respectively) compared with the reference beam (BN). Better enhancement in both cracking and ultimate torques was recorded for the composite beams of 150 mm plate height (from 200% to 250% and 204% to 278% for steel plate thickness 1.5 and 2 mm respectively) in comparison with the reference beam (BN). It can be noticed that all beams connected to steel plate of 150 mm in height were stronger than the other beams, while the weakest beam was the reference beam without a steel plate (BN).

Tests for the torque versus angle of twist, which explains the effect of the steel plates’ height on the tested beams, were constructed, as shown in Figures 7 and 8. At higher loads, it was found that, for the same loading values, the increase in the plate height will decrease the angle of twist significantly due to increasing the area under the curve (stiffness) and the energy absorption capacity (ductility). A possible reason for this increment in the stiffness and ductility is due to contribution of the steel plate, as steel is a ductile material and therefore, as the height of the steel plate was increased, the ductility and stiffness of the concrete beams increase, which is a very desirable feature for safer design. At the same torsional moment (7.5 KN.m), the angle of twist was decreased from 0.294˚ to 0.196˚; this decrease in the angle of twist is due to the effect of increasing the steel plate area, as shown in Figure 7.

It was concluded from Figures 9 and 10 that the steel plate height made a good contribution for decreasing the longitudinal elongation of the composite beams. From all the results, the height of steel plate increased the torsion capacity to the beams. Since, the height of plate increases the stiffness of beam, and that make the beams give the high strength with applied torque.

Table 3. Cracking Torque Results for the Experimentally Tested Beams

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>Plate Thickness mm</th>
<th>Plate Height mm</th>
<th>Bolt Spacing Mm</th>
<th>Cracking Torque (KN.m)</th>
<th>Angle of Twist (deg./m)</th>
<th>Percent of Torque increase %</th>
</tr>
</thead>
<tbody>
<tr>
<td>BN</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>2.50</td>
<td>0.261</td>
<td>------</td>
</tr>
<tr>
<td>B1</td>
<td>2</td>
<td>150</td>
<td>50</td>
<td>8.75</td>
<td>0.335</td>
<td>250</td>
</tr>
<tr>
<td>B2</td>
<td>1.5</td>
<td>150</td>
<td>50</td>
<td>7.5</td>
<td>0.376</td>
<td>200</td>
</tr>
<tr>
<td>B3</td>
<td>2</td>
<td>100</td>
<td>50</td>
<td>7.5</td>
<td>0.294</td>
<td>200</td>
</tr>
<tr>
<td>B4</td>
<td>1.5</td>
<td>100</td>
<td>50</td>
<td>5.62</td>
<td>0.32</td>
<td>126</td>
</tr>
</tbody>
</table>

Table 4. Ultimate Torque Results for the Experimentally Tested Beams

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>Plate Thickness mm</th>
<th>Plate Height mm</th>
<th>Bolt Spacing</th>
<th>Ultimate Torque (kN.m)</th>
<th>Angle of Twist (degree/m)</th>
<th>Percent of Torque increase %</th>
</tr>
</thead>
<tbody>
<tr>
<td>BN</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>5.62</td>
<td>5.945</td>
<td>------</td>
</tr>
<tr>
<td>B1</td>
<td>2</td>
<td>150</td>
<td>50</td>
<td>21.25</td>
<td>6.33</td>
<td>278</td>
</tr>
<tr>
<td>B2</td>
<td>1.5</td>
<td>150</td>
<td>50</td>
<td>17.125</td>
<td>4.053</td>
<td>204</td>
</tr>
<tr>
<td>B3</td>
<td>2</td>
<td>100</td>
<td>50</td>
<td>18.125</td>
<td>4.606</td>
<td>222</td>
</tr>
<tr>
<td>B4</td>
<td>1.5</td>
<td>100</td>
<td>50</td>
<td>14.125</td>
<td>3.433</td>
<td>151</td>
</tr>
</tbody>
</table>
Figure 6. Torque-Twist Relationship of R.C Beams of 2 mm Plate Thickness

Figure 7. Torque-Twist Relationship of R.C Beams of 1.5 mm Plate Thickness

Figure 8. Torque-Longitudinal Elongation Relationship of R.C Beams of 2 mm Plate Thickness
3.2. Effect of Steel Plate Thickness on Torsional Behavior

Figures 11 to 14 show the effect of plate thickness on the beams’ torsional behavior. It was obvious from the general test results that the overall properties of the beams are enhancing when the plate thickness increases, where both the cracking and ultimate torque increased as the thickness of the steel plate increased. The tested beams attached to steel plate of 1.5 mm thickness exhibited an increase in cracking and ultimate torques (from 124% to 200% and 151% to 204% for the 100 and 150 mm plate height respectively) compared with the reference beam (BN). Better enhancement in both cracking and ultimate torques was recorded for the composite beams of 2 mm plate thickness (from 200% to 250% and 222% to 278% for the 100 and 150 mm plate height respectively) in comparison with the reference beam (BN). It can be noticed that all the composite concrete-steel beams connected to 2 mm-thick steel plate were stronger than the other composite beams connected to 1.5 mm-thick steel plate.

The influence of the steel plate's thickness on the torque-twist response is shown in Figures 11 and 12. In these figures it founds that, during the early loading stage, the torque-twist behavior of the composite beams was almost the same but with little decrease in the angle of twist due to the increase in plate thickness, and this continued until the applied torque equated the first crack torque approximately. Then, the angle of twist would decrease significantly with the increase in loading due to the global stiffness and ductility of the composite beams increasing with an increase in the steel plate's thickness. As seen in Figure 11, when increasing the steel plate’s thickness from 1.5 mm in B2 to 2 mm in B1 at the same torsional moment (7.5 KN.m), the angle of twist was decreased from 0.367° to 0.196°.
The relationship between longitudinal strain and torsional moment is shown in Figures 13 and 14. Based on these figures, it can be concluded that the steel plate made a good contribution for decreasing the longitudinal elongation of the tested beams. From all the results, the thickness of steel plate increased the torsion capacity to the beams. Since, the thickness of plate increases the stiffness of beams, and that make the beams give the high strength with applied torque.

Figure 11. Torque-Twist Relationship of R.C Beams of 100 mm Plate Height

Figure 12. Torque- Longitudinal Elongation Relationship of R.C Beams of 150 mm Plate Height

Figure 13. Torque- Longitudinal Elongation Relationship of R.C Beams of 100 mm Plate Height
4. Conclusions

Based on the results obtained from the experimental work, the following conclusions are presented.

- It can be concluded that the use of steel plates that are mechanically attached to both sides of the reinforced concrete beams under torsional load enhances the overall properties of these members.
- It was shown that, as the thickness of the 100mm steel plate connected to the reinforced concrete beam increased from 1.5 mm to 2 mm, the ultimate torsional moment increased by an average of 28%; also, the cracking torsional moment increased by an average of 33.5%.
- It was shown that, as the height of the steel plate connected to the reinforced concrete beam increased from 100 mm to 150 mm, the ultimate torsional moment increased by an average of 21.2%; also, the cracking torsional moment increased by an average of 33.5% for the 1.5mm steel plate thick.
- The global stiffness and ductility of the composite beams increased significantly as the thickness and the height of the steel plate increased.

5. Conflicts of Interest

The authors declare no conflict of interest.

6. References


A Quantitative Approach to Prioritize Sustainable Concrete

L. Sudheer Reddy a, A. Suchith Reddy b, S. Sunil Pratap Reddy a*

a Department of Civil Engineering, Kakatiya Institute of Technology & Science Warangal (KITSW), Telangana, India.

b Department of Civil Engineering, National Institute of Technology Warangal (NITW), Telangana, India.

Received 21 July 2019; Accepted 29 October 2019

Abstract

Cement industry consumes high energy and produces major emissions to the environment. In order to reduce the effects (environmental impact, energy, and resources) caused by conventional materials, various by-products and pozzolonic material are used to achieve sustainable concrete. Assessing the concrete performance based on multiple conflicting attributes is decisive and compelling. It is difficult to choose an alternative among the Supplementary Cementitious Materials (SCM) considering a set of quantitative performance attributes. Hence, the present study utilizes the theories of decision making to prioritize an alternative environmentally and technologically. The purpose of the present study is to observe the sustainable performance of five different concretes made of OPC, Fly ash, GGBS, Metakaolin and Composite Cement for a particular grade of concrete. The study has considered workability, strength attribute (compressive strength, split tensile and flexural strength) and durability attribute (Sorptivity and RCPT) at their respective optimum replacements. To prioritize an alternative material considering quantitative attributes, Technique for Order of Preference by Similarity to Ideal Solution (TOPSIS) is utilized. From the results, it is observed that considering all attributes, flyash based concrete has higher performance and is prioritized among others. The developed approach facilitates the decision-makers in the selection of a sustainable alternative.

Keywords: Sustainable Concrete; Supplementary Cementitious Material; Multi-Criteria Decision Method; Environment; TOPSIS.

1. Introduction

In developing countries like India, the population is increasing at an asymptotic rate, thus there is a demand for all types of infrastructure facilities. The building industry is growing at a faster rate by consuming the major natural resources resulting in higher carbon footprint [1]. India is the fourth-largest emitter of CO₂, where the major contributor of it is the energy sector with the construction industry being a subset of it [2]. Thus, there is an urgent need to shift our thoughts towards sustainability. To attain sustainability in the construction industry, materials play a crucial part. Selection of suitable material which serves the purpose of the application without degrading the environment leads to sustainable construction. Currently, the construction industry is utilizing industrial by-products and waste materials to decrease the potential impact on natural/non-renewable resources [3]. Choosing appropriate material at the design stage will facilitate to reduce the impacts on the environment [4]. Selection of suitable sustainable material for construction will minimize the impacts, energy consumption and waste production. This will also increase the potential utility for future generations [5]. Therefore, by implementing the principles of sustainability in the construction industry by neglecting conventional practices will certainly achieve an ecological balance between future and present requirements [6].

*Corresponding author: sunilpratap@yahoo.com

http://dx.doi.org/10.28991/cej-2019-03091434

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
According to the United Nations, World Commission on Environment and Development has defined sustainable development as “development which meets the needs of present generations without compromising the ability of future generations to meet their own needs” [7]. Furthermore, the influence of materials is observed not only on the environment but also on social and economic aspects [8, 9]. Assessing the material performance to choose the right/appropriate material for achieving sustainability is a crucial aspect in subjectivity. Evaluating the material performance based on one or two parameters and selecting it is not beneficial [10]. Assessment based on a combination of conflicting attributes and constructive attributive is desirable. Although most of the studies concluded that the characteristic performance of a material based on physical, mechanical and durability properties individually [11, 12]. Different materials may perform differently with respect to a single attribute. To choose an optimal material and achieve the desired results, the requirements should be robust enough to achieve the required performance. For example, in the case of concrete, the cost should be reasonable, should be durable and also obey sustainable design principles.

Concrete is the second maximum consumed material next to the water. It is expected that the demand for concrete will rise up to 16 billion ton per year by 2050 [13]. The use of concrete cannot be avoided due to its unique features and advantages, but the impacts caused by concrete can be reduced by producing concrete by focusing more on sustainable aspects like the use of right materials, methods, and technologies [14, 15]. In the recent past literature, most of the researchers are using by-products like Fly ash, Ground Granulated Blast Furnace Slag, Metaakolin, Silica fume, Risk Husk Ash and Composite Cement, etc., in the place of conventional cement material [12, 13]. Thus a situation has arisen, where the selection of suitable cementitious material based on the combined effect of workability, strength, design performance and cost has become difficult. The use of the decision-making theories thus facilitates to prioritize the best sustainable material. Most of the studies proposed various methods in the literature based on the Multi-Criterion Decision Making (MCDM) theories [14, 15]. Every method has its own limitation and applicability on the decision problem. Different applications need different types of concretes, but it is a challenge to decrease the cement content to reduce the environmental effect of them [19, 20]. However, selecting suitable materials for the design of concrete involves various attributes like physical properties, workability, strength, environmental performance, fire resistance, durability aspects, cost, etc., has to be considered concurrently instead of considering only single attribute at a time. In addition, a single attribute cannot judge the performance of concrete with distinctive properties and satisfy the desired properties. To evaluate any material performance, it is necessary to frame a set of significance attribute/criteria. Most of the studies have considered environmental and economic aspects related attributes and none of them talks about the technical aspects and assess the quantitative concrete sustainability [13]. In the present study, an attempt is been made to integrate Workability (W), Compressive Strength (CS), Split Strength (SS), Flexural Strength (FS), Sorptivity (S), Rapid Chloride Penetration Test (RCPT) and Life Cycle Cost (LCC) of a particular grade of concrete in prioritising sustainable material alternative. It is difficult to prioritize the mixes considering various test results developed from various percentage replacements. Hence, the objective of the study is to evaluate the sustainable performance of five different Supplementary Cementitious Material (SCM) including OPC, Fly ash based PPC, Slag based PPC, Metaakolin and Composite Cement using Technique for Order Preference by Similarly to Ideal Solution (TOPSIS) at their respective optimum replacement levels.

2. Research Methodology

Selection of the best material considering several parameters makes the material selection complex and tedious problem [21]. Multiple attributes and material alternatives made the present study to utilize the MCDM technique in selecting the material alternative. A framework has been developed to choose the best binder material considering nine quantitative attributes for achieving sustainable concrete utilizing the TOPSIS method. The five binder material alternatives like Ordinary Portland Cement (OPC), Fly ash (PPC- F), GGBS (PPC- S), Metakaolin (M), and Composite Cement (CC) are considered in producing M20 grade of concrete with 0, 20, 20, 10 and 100% replacement respectively. Figure 1 shows the methodology in carrying the material selection using the TOPSIS method with respect to the decision problem. As the sustainability and durability go hand in hand, it is vital to consider durability aspects in assessing the quantitative performance of concrete along with strength parameters, the basic durability aspect for any type of concrete is related to water absorption through capillary action i.e. Sorptivity. Similarly, the concrete deterioration due to chloride ingress is another importance aspects of durability. The present study considered nine attributes pertaining to fresh properties of concrete, hardened mechanical properties and durability aspects stated as, Workability (W), Compressive strength (CS) for 28, 56 and 90 days, Split tensile (SS) Flexural Strength (FS), Sorptivity (SR), Rapid Chloride Penetration Test (RCPT) and Life Cycle Cost (LCC) in evaluating the quantitative performance of a concrete. Based on the applicability of concrete, the concrete durability aspects are selected. The sorptivity and RCPT is performed to observe the durability performance involved with various SCM’s at their optimum replacement levels for M20 grade of concrete. The fineness of material alternatives OPC, PPC – F, PPC- S, M and CC are found to be less than 10% by weight of material and specific gravities are observed to be 3.05, 2.14, 2.65, 2.96, and 2.86 respectively.
The Technique for Order of Preference by Similarity to Ideal Solution (TOPSIS) is used to calculate the relative closeness of alternative solution with the positive and negative ideal solution. It is most widely used MCDM which works on the principles of finding the optimized solution considering conflicting criteria[22]. In the TOPSIS method, the selection of material is based on the attributes having equal importance. However, the weight to attributes can be evaluated using other MCDM techniques and integrate with the TOPSIS method in selecting the best alternative[23]. The present study explores the concept of TOPSIS technique in selecting sustainable building material considering nine quantitative attributes. The following steps are involved in evaluating the best alternative.

**Step 1:** Creating the decision matrix with ‘p’ number of criteria and ‘q’ number of alternatives. The performance of $i^{th}$ alternative with regard to the $j^{th}$ attribute is expressed as $x_{ij}$ and the matrix is formed as shown below:

$$d_{ij} = \begin{bmatrix} x_{11} & x_{12} & \cdots & x_{1n} \\ x_{21} & x_{22} & \cdots & x_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ x_{m1} & x_{m2} & \cdots & x_{mn} \end{bmatrix}$$

**Step 2:** Normalize the decision matrix to obtain non-dimensional values. The normalized matrix $N_{ij}$ is obtained by the Equation 1.

$$A_{ij} = [a_{ij}]_{mn} = \frac{x_{ij}}{\sqrt{\sum x_{ij}^2}}$$

Then the normalized matrix would be $A_{ij} = \begin{bmatrix} a_{11} & a_{12} & \cdots & a_{1n} \\ a_{21} & a_{22} & \cdots & a_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ a_{m1} & a_{m2} & \cdots & a_{mn} \end{bmatrix}$

**Step 3:** Evaluate the Positive Ideal Solution (PIS) $A_{ij}^+$ and Negative Ideal Solution (NIS) $A_{ij}^-$ by the Equations 2a and 2b:
\[
A_j^+ = (A_1^+, A_2^+, A_3^+ \ldots A_n^+) = \left\{ \begin{array}{l}
\text{Max}_{i,j} \forall j \in k \\
\text{Min}_{i,j} \forall j \in k'
\end{array} \right.
\]
\[
A_j^- = (A_1^-, A_2^-, A_3^- \ldots A_n^-) = \left\{ \begin{array}{l}
\text{Min}_{i,j} \forall j \in k \\
\text{Max}_{i,j} \forall j \in k'
\end{array} \right.
\]

Where \( k \) is a set of benefit attribute whereas \( k' \) is a set of cost attribute.

**Step 4:** Compute distance between target ideal solution to PIS \((D^+)\) and target ideal to NIS \((D^-)\) by Equation 3a and 3b;

\[
D_i^+ = \sqrt{\sum_{j=1}^{n} (A_{ij} - A_j^+)^2} \quad \forall j \in n; \quad \forall i \in m
\]  
(3a)

\[
D_i^- = \sqrt{\sum_{j=1}^{n} (A_{ij} - A_j^-)^2} \quad \forall j \in n; \quad \forall i \in m
\]  
(3b)

**Step 5:** Calculate the Relative Closeness Coefficient \((CC)\) for each alternative from the ideal solution from Equation 4. Higher the closeness coefficient, better is the material sustainability.

\[
CC_i = \frac{D_i^-}{D_i^+ + D_i^-}
\]  
(4)

### 3. Results and Discussion

The study used the TOPSIS approach to the decision problem of selecting the best binder material alternative considering nine parameters. The findings of the study optimized the positive attributes and minimized negative attributes. The study considered a fresh property like Workability \((W)\), hardened mechanical properties like Compressive Strength \((CS)\) with respect to the age of curing for 28, 56 and 90 days, Split tensile \((SS)\) Flexural Strength \((FS)\), durability properties like Sorptivity \((SR)\), RCPT and LCC. In the present study, the prevailing rates of material, labor and transportation, and manufacturing in South India have been considered. The Tables (1 – 6) illustrate the results of the TOPSIS method. The results obtained from experimental investigation for nine properties of the concrete made of different SCM’s is considered has a decision matrix and are shown in Table 1. Based on the step by step approach described in section 4 the decision problem has been resolved for selecting a best binder material alternative.

**Step 1:** Formulate the decision matrix considering attributes and alternative. The present study considered nine attributes and five alternatives (Table 1).

<table>
<thead>
<tr>
<th>Attributes</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>C6</th>
<th>C7</th>
<th>C8</th>
<th>C9</th>
</tr>
</thead>
<tbody>
<tr>
<td>W (mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28 Days</td>
<td>25.20</td>
<td>30.08</td>
<td>31.39</td>
<td>2.15</td>
<td>5.96</td>
<td>0.458</td>
<td>2100</td>
<td>9650</td>
<td></td>
</tr>
<tr>
<td>56 Days</td>
<td>28.34</td>
<td>30.52</td>
<td>31.16</td>
<td>2.08</td>
<td>4.81</td>
<td>0.342</td>
<td>1248</td>
<td>8350</td>
<td></td>
</tr>
<tr>
<td>90 Days</td>
<td>27.23</td>
<td>34.88</td>
<td>37.49</td>
<td>2.56</td>
<td>5.92</td>
<td>0.468</td>
<td>1648</td>
<td>8950</td>
<td></td>
</tr>
<tr>
<td></td>
<td>26.16</td>
<td>32.70</td>
<td>35.06</td>
<td>1.52</td>
<td>4.82</td>
<td>0.621</td>
<td>1356</td>
<td>9129</td>
<td></td>
</tr>
<tr>
<td></td>
<td>28.34</td>
<td>30.08</td>
<td>34.88</td>
<td>2.49</td>
<td>6.41</td>
<td>0.547</td>
<td>1895</td>
<td>8432</td>
<td></td>
</tr>
</tbody>
</table>


**Step 2:** Normalizing the decision matrix to a non-dimensional unit matrix. The normalization technique is carried out using Equation 1 and represented in Table 2.
Step 3: The Positive Ideal Solution (PIS) and Negative Ideal Solution (NIS) is evaluated using the Equations (2a and 2b) and are represented in Figure 2. The study observed that the attributes W, CS, SS and FS are to be maximized (beneficial attributes) and SR, RCPT, and LCC are minimized (cost attributes).

Step 4: Compute the distance between the target ideal solution to PIS ($D^+$) and target ideal to NIS ($D^-$) by Equation (3a and 3b) as shown in Table 3.

Step 5: Compute the Closeness Coefficient of each alternative with respect to PIS and NIS using Equation 4 and the findings are represented in Table 4. Higher the Closeness Coefficient (CC) better is the material performance. The findings of the study reveal that the positive and negative values for each attribute will either be converging or diverging towards the ideal solution line (auxiliary zero line) (Figure 2). For example, the criteria Compressive Strength (CS) should be maximized, which is a positive aspect for any alternative, and negatively it should be minimized. Similarly, criteria sorptivity (C7) should be minimized, which is a positive aspect for any alternative and negatively it should be maximized. In the case of criteria C7, the positive and negative values are diverging and for criteria CS, it is observed to be converging towards the ideal solution. The ideal solution (Alternative) is selected based on criteria nearest distance to Positive Ideal Solution (PIS) and farthest Negative Ideal Solution (NIS). Considering equal importance to criteria, from Figure 3, it can be observed that the material alternative (A2) PPC flyash based (PPC-F) is having nearest PIS and farthest NIS. The order of preference for selecting the sustainable binder material alternative based on CC values and is found to be A2 > A3 > A5 > A1 > A4 (Table 4).
4. Conclusions

The present study explored the use of decision-making method TOPSIS in selecting the best sustainable alternative considering quantitative attribute (Technological aspect).

The study has considered nine quantitative attributes, fresh property- Workability, hardened property- Compressive Strength (28, 56 and 90 days), Split tensile and Flexural Strength, durability property - Sorptivity, RCPT and Life Cycle Cost (LCC) in prioritizing the best sustainable material alternative. The following conclusions were drawn for selection of the best sustainable alternative.

- The study has considered nine quantitative attributes i.e., Workability, Strength (28, 56, 90) days, split tensile, Flexural strength, Sorptivity, RCPT and Lifecycle cost for selecting the best alternative.
- Amongst five SCM’s, flyash based PPC is prioritized with the highest closeness coefficient of 0.67 whereas Metakaolin has achieved the leastvalue of 0.38.
- The order of priority for selection of sustainable binder material alternative is found to be Flyash based PPC, Slag based PPC, Composite cement, OPC and Metakaolin.
- The approach explored in the study will facilitate the designers to take decisions in selecting the best sustainable material among a pool of available alternatives

5. Acknowledgements

The authors would sincerely like to thank the management of Kakatiya Institute of Technology (KITSW-Autonomous), Warangal, Telangana, Warangal for providing the testing facilities and their constant support.

6. Conflicts of Interest

The authors declare no conflict of interest.
7. References


Structural Characteristics of Developed Sustainable Lime-Straw Composite

Sajid Kamil Zemam a*, Sa'ad Fahad Resan a, Musab Sabah Abed a

* Civil Engineering Department, Engineering College, University of Misan, Amarah, Iraq.

Received 25 June 2019; Accepted 29 September 2019

Abstract

Construction materials made of renewable resources have promising potential given their low cost, availability, and environmental friendliness. Although hemp fibers are the most extensively used fiber in the eco-friendly building sector, their unavailability hinders their application in Iraq. This study aimed to overcome the absence of hemp fiber in Iraq and develop a new sustainable construction material, strawcrete, by using wheat straw and traditional lime as the base binder. A comparable method of developing hempcrete was established. The experimental program adopted novel Mixing Sequence Techniques (MSTs), which depended on changing the sequence of mixed material with fixed proportions. The orientation of the applied load and the specimen’s aspect ratio were also studied. The mixing proportion was 4:1:1 (fiber/binder/water) by volume. Results showed that the developed strawcrete had a dry unit weight ranging from 645 kg/m³ to 734 kg/m³ and a compressive strength ranging from 1.8 MPa to 3.8 MPa. The enhanced physical and strength properties varied with the MST and loading orientation. The properties of the developed hempcrete were compared with those of strawcrete.

Keywords: Hempcrete; Strawcrete; Wheat Straw Fiber; Mixing Sequence Technique; Compressive Strength; Loading Orientation; Strength Rating.

1. Introduction

In recent years, the tendency for designing low-environmental-impact buildings to meet the requirement of ecosystems has emphasized on the global use of bio-aggregate-based concretes. The term bio-aggregate concretes refers to the mixture of binders (lime, clay, plaster, and cement) and natural fibers (hemp, straw, flax, bamboo, and animal hairs) [1]. In this context, the use of eco-friendly concrete such as hempcrete [2], wood-concrete [3], papercrete [4], and mud-concrete [5] has been growing considerably. Hempcrete is most widely used in the field of green construction owing to its remarkable environmental quality as a non-CO₂ producer [6, 7]. Furthermore, walls made of hemp–lime composite exhibit better sound absorption and thermal isolation than conventional concrete walls [8]. Hempcrete had been introduced in the early 90s in France by using the matrix of lime and hemp shiv particles [9]. From the construction point of view, hempcrete, similar to several biomass concretes, is predominantly non-load bearing material; nevertheless, its strength is important to provide the solidity to hold its own weight [10, 11].

The strength of hempcrete significantly depends on the binder type, density, and morphology of hemp fiber. Hempcrete density varies with the applied tamping effort [12]. Four levels of densities are usually identified to be vary from very light density to high density [13]. Extensive research has been undertaken to determine the factors affecting hemp concrete strength. E. P. Aigbomian [14] stated that the compressive strength of hemp concrete varies with mixture

*Corresponding author: sajid.kamil@uomisan.edu.iq

http://dx.doi.org/10.28991/cej-2019-03091435

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
proportions from 0.02 MPa to 1.22 MPa. Moreover, the binder added with 10%–25% Portland cement can improve the compressive strength of hempcrete and increase its density [13]. Previous studies have focused on enhancing the mechanical properties of hempcrete by selecting the raw materials, percentage, and types of binders; furthermore, the importance of the applied tamping effort and particle size distribution of bio-aggregates has been highlighted [15]. Thus, hempcrete has paved the way for the utilization of agro-fiber in the construction industry. In many countries and in Iraq, hemp fiber is unavailable because of legal issues, thereby obstructing the use of hempcrete. Abundant agricultural by-products were adopted as alternatives to hemp fibers for use in the construction sector to overcome this issue. In this context, rice husk fiber was used as an alternative to hemp fiber to develop a new isolation building envelop [16].

Likewise, woodcrete was developed using sawdust and lime as base binder; woodcrete a new building material that is capable of withstanding loads similar to that hempcrete [14]. Growing attention has been directed to straw fiber as building materials in the past decades. In this context, straw bale building has significantly contributed to the design of low-carbon and energy-efficient building [13] given its thermal and acoustical advantages [17]. Straw fiber is usually a by-product of annual growing crops; thus, it is cheap and obtainable in most countries. Wheat is the major source of straw fiber. Iraq occupies the 31st position in global wheat production. The average wheat production in Iraq is 3,753,111 t from 2014 to 2016 [18]. Therefore, the use of straw fiber for building purposes provides a sustainable and ecological approach of recycling [14]. Reusing abundant wheat straw fiber in the present study as an alternative of hemp fiber for bio-concrete closes the gap of hemp absence in Iraq.

Discussion about the importance of mixing sequence and its influence on strength characteristics of bio-aggregate base concretes is few [9]. In this context, some authors suggested to preliminary blend water and binder in a pan mixer to produce a homogenous slurry; afterward, the bio-aggregate was added and further mixed until full uniformity was reached [15]. Other authors proposed a mix by pre-wetting the bio-aggregates, and lime-based binder was added subsequently [16]. All of the adopted sequences were undertaken as a mixing procedure without considering their effect on strength characteristics. Therefore, this paper first proposed straw fiber as an alternative to hemp and examined the significant influence of mixing sequence on the physico-mechanical properties of bio-aggregate-based concretes. Loading orientation with respect to tamping layer was also studied.

2. Experimental Program

The experimental work was performed at the Construction Laboratory of Engineering Faculty at Misan University. The main variables considered in this study were related to Mixing Sequence Techniques (MST), loading direction with respect to orientation of tamping layers, casting age effect, and sample aspect ratio.

2.1. Materials

Wheat straw fiber (WSF) was chosen as a bio-aggregate because of its abundant supply in Iraq. On the basis of previous research carried out on hemp shiv slices, fiber size (length) has been found as a function of density and strength [19]. Strawcrete, the straw fiber in this study, was used in its natural morphology to determine the effect of mixing sequence on the strength properties of the developed bio-aggregate. Furthermore, Joseph Williams stated that loading orientation does not depend on fiber size [15]. The fiber size distribution ranges from 5 mm to 50 mm as shown in Figure 1. Wheat straw has a number of uses as fuel, livestock bedding, feed, fodder, and in thatching. In the construction sector, the use of WSF in Iraq has a historical value as a major additive for improving unfired clay building units called adobe [20] of ancient clay house. Locally available lime powder is used throughout this work as base binder in which it is conform to BS 890 Class B [21]. Moreover, 50% of binder was included with ordinary Portland cement to gain extra strength for the developed bio-composite in this study as recommended by literature [13]. Chemical analysis and the main compounds of the used cement aside from the physical properties conformed to the Iraqi specification number (5/1984) [22] and in accordance with the ASTM C191 [23].

2.2. MSTs

The sequence of mixing in bio-aggregate concrete has not been specified until now. In the present study, two different sequences have been designed as shown in Figures 2 and 3. MST-I in Figure 2(a) represents the mixing sequence in which the binder has been pre-mixed with water to produce a semi-thick slurry and then added to the dry fiber. The materials were mixed gently. The casting of bio-composite materials requires a specific effort to ensure an acceptable density for the produced samples. MST-II in Figure 2(b) shows the mixing sequence in which the total water has been pre-mixed with dry straw fiber. Then, the binder was added in the form of powder and mixed together. The binder was mixed with wheat straw fiber with a fiber/binder ratio of 4:1 as recommended in previous studies [24]. Table 1 shows the material proportions for the two used techniques.
Figure 1. Straw fiber size distribution

Figure 2. Mixing Sequence Techniques (MST)

Figure 3. Mixing procedure
Figure 4. Flow chart of the research methodology

Table 1. Material proportions of the adopted mixes

<table>
<thead>
<tr>
<th>No.</th>
<th>Designation</th>
<th>Mixing sequences</th>
<th>Lime kg/m²</th>
<th>Cement kg/m³</th>
<th>Binder kg/m³</th>
<th>Water/binder ratio</th>
<th>Water kg/m³</th>
<th>Wheat straw Binder ratio</th>
<th>Wheat straw fiber kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MST-I</td>
<td>Binder–water–fibre</td>
<td>100</td>
<td>108</td>
<td>208</td>
<td>1:1.25</td>
<td>166</td>
<td>1:2.85*</td>
<td>73</td>
</tr>
<tr>
<td>2</td>
<td>MST-II</td>
<td>Fibre–water–binder</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.3. Fiber Orientation and Loading Modes

The same compacted effort was used for both mixing techniques. The number of tamping for each layer was designed as 20 with three layers for each sample. The weight of the tamping tool was 2 kg, and the drop height was approximately 30 cm. Thus, the constant applied effort was 105 kN.m/m³. The tamping direction forced wheat straw fibers within the matrix to be oriented to the plane of the tamped layers. As loading was applied in a specified orientation, we investigated two basic cases to the study matrix anisotropy. These were the loading normally applied to the tamping layer orientation (loading mode 1) and along it (loading mode 2). Figure 5 shows the configuration of the tamped layers and their respective loading modes.

Figure 5. Configuration of the tamped layers and their respective loading modes
2.4. Sample Fabrication and Test Setting

Various compressive and flexural tested specimens were fabricated depending on the previously described mixing techniques and tested in the assigned two different loading modes. The duration of specimen testing was 56 days. A compression machine (ELE) was used to test the specimens for compressibility, whereas a 10 t hydraulic jack was used for flexural tested specimens. Testing load is usually directly applied using a relatively rigid steel plate at a maintained constant rate of 10 kg/s by using equal increments at each step. The mechanical dial gauges with accuracy of 0.01 mm were placed to measure the specimen’s deformation, whereas applied loading was recorded using a calibrated load cell. Figures 6 and 7 show the fabricated specimens and described test arrangement. The compression specimens were encoded using a simplified designation, “ABCi” as follows:

- Denoted aspect ratio, S for cubes and P for prisms
- Denoted applied loading direction, N for mode 1 and P for mode 2
- Denoted mixing sequence technique, T1 for MST-I and T2 for MST-II

![Fabricated specimens](image1)

![Test arrangement](image2)

3. Results and Discussion

The experimental results of various specimens related to the two adopted matrices showed that although these matrices had the same material proportions and tampering efforts, the specific mixing method or specific wheat fiber orientation affected their mechanical properties and structural performance. Table 3 shows the mechanical and structural characteristics of the developed strawcrete specimens discussed throughout this study. The mechanical properties and structural characteristics of the developed strawcrete specimens with different MSTs and WSF orientation are listed in Table 1. The specimens fabricated using MST-II with the fiber orientation mode 1 exhibited high compressive strength (3.8 MPa), fracture strength (0.69 MPa), and flexural capacity (4.66 MPa). The corresponding compressive strength rating (2.83 MPa) and flexural strength rating (4.4) maintained high ratings among others as the unit weight was slightly affected by the mixing technique. The bulk density changed from 645 kg/m³ to 734 kg/m³. For various mixing techniques or fiber orientation, high-strength specimens exhibited softening behavior assigned by the modulus of elasticity dropping. Specimen compressibility, which could be indicated by Poisson ratios, confirmed the fiber orientation mode 1 as a matrix of high compressibility. The assigned negative Poisson ratio could be affected by large out-of-plane buckled layers within the matrix of fiber orientation 2. The modulus of elasticity usually increased as the Poisson ratio decreased, thereby resulting in extreme anisotropy. Poisson ratio, ν, and elastic modulus, E, of different specimens are summarized in Table 2.
Table 2. Structural characteristics of the developed strawcrete specimens

<table>
<thead>
<tr>
<th>Group</th>
<th>MST</th>
<th>Bulk Density, kg/m³</th>
<th>WSF Orientation</th>
<th>Modulus of Elasticity, MPa</th>
<th>Poisson Ratio, N</th>
<th>Compressive Strength, MPa</th>
<th>Compressive Strength Rating</th>
<th>Fracture Strength, MPa</th>
<th>Flexural Capacity, kN.m</th>
<th>Flexural Strength Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MST-I</td>
<td>645</td>
<td>Mode1</td>
<td>7.1</td>
<td>0.053</td>
<td>1.8</td>
<td>1.94</td>
<td>0.22</td>
<td>1.49</td>
<td>1.6</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td>Mode2</td>
<td>10.54</td>
<td>0.029</td>
<td>0.34</td>
<td>0.37</td>
<td>0.09</td>
<td>0.61</td>
<td>0.656</td>
</tr>
<tr>
<td>3</td>
<td>MST-II</td>
<td>734</td>
<td>Mode1</td>
<td>10</td>
<td>0.07</td>
<td>3.8</td>
<td>2.83</td>
<td>0.690</td>
<td>4.66</td>
<td>4.40</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td>Mode2</td>
<td>14.84</td>
<td>0.04</td>
<td>0.697</td>
<td>1.66</td>
<td>0.288</td>
<td>1.94</td>
<td>1.83</td>
</tr>
</tbody>
</table>

3.1. MST Effects

The second-adopted MST (MST-II) was assigned as the most efficient proper technique because for the same loading orientation (normal to tamping layers), the compression strength of the specimens prepared using MST-II were 3.8 MPa (for SNT2) against 1.8 MPa (for SNT1) and was prepared via MST-I. Figure 8 shows the stress–strain trending in the scope of the MST effect. Figure 9 shows the loading–time trending in terms of the MST effect. For the loading orientation distinguished using parallel to tamping layers, the stress–strain behavior shows a ductile behavior in comparison with the normal mode with humble compression strength although the specimens prepared using MST-II were advancing in developing strength. Figures 10, shows the stress–strain trending of MST effects in the scope of tamping orient mode 2.
3.2. Fiber Orientation and Loading Mode Effect

Two distinguished loading methods were considered in this study. In the first method, one load was applied normally to the tamping layers, and the second one was applied along the tamping layers. The normal mode was assigned as the excellent mode to utilize the full strength capacity of the developed matrix. For the same sequence technique, the compression strength dropped from 3.8 MPa to 0.9 MPa as the loading mode changed from normal to parallel. Figure 11 shows the stress–strain trend in the scope of the tamping orientation effect in which the specimens subjected to loading parallel to tamping layout exhibited comprehensive mechanically softer response than those undergoing normal loading mode (mode 1). Sensitizing the tamping layer orientation for the applied loading direction highly affected the stress concentration within and around the bonded layers; these layers dominated the elasticity of the entire matrix and resulted in similar plates buckling along the specimen’s depth.

3.3. Aspect Ratio Effect

In any construction materials, the aspect ratio of dimension related to the tested specimens affect the compression strength. The compressive strength decreased from 3.8 MPa to 2.807 MPa as the aspect ratio doubled. The same observation indicated that for both loading modes, the reduction rates were 0.74 and 0.84 for normal and parallel loading modes, respectively, as listed in Table 3. Figures 12 and 13 show the stress–strain trend in the scope of the aspect ratio effect for normal and parallel loading modes, respectively.
4. Failure Modes

Under compression loading, the deformed bonds across the concrete tamping layers within the section were more pronounced in loading mode 2 than that in loading mode 1. Thus, two different failure modes have been assigned as shown in Figure 14. The first is related to the first loading method, which is distinguished by developing a crushing layer across the section because of the crushing of the bonding structure under compressive stresses. The specimens tested by second loading fashion failed as the buckled bands of discrete plates. This mode of failure could be attributed to bond separation because of the developed shear stress between layers previous to buckling, and the specimens of aspect ratio (2) assigned buckling trending observed. The specimens under bending produced numerous micro-cracks after small movements. These micro-cracks might be recrystallized during the reaction of free lime, thereby effectively...
self-healing the affected region. With a large movement, the flexural failure mode distinguished by the main crack continued until failure with the absence of any secondary cracks. Figure 15 confirmed the brittleness challenge of the developed matrix.

Figure 14. Fiber-oriented effect on failure mode

(a) Front side view

(b) Base side view

Figure 15. Flexural mode configuration of prism specimens distinguished with the main crack

5. Proposed Strawcrete Vs. Customary Hempcrete

The mechanical properties of the developed strawcrete had been compared with those of customary hempcrete blocks made from a mixture of lime and hemp shives, which were investigated by Elfordy [25] and manufactured via projection of mixture of 34% of a lime-based binder (70% of hydrate lime, 15% of pozzolanic material, and 15% of hydraulic binder), 16% hemp shives, and 50% water. Table 4 summarizes the specified samples of corresponding properties of investigated hempcrete for comparison with those presented in the results of the current study. The analysis confirmed the high similarity of elasticity response of strawcrete with that of customary hempcrete. Strawcrete showed positive improvement of gained compressive strength, extended to 1.8 for MST I against 0.18 MPa for the same mixing technique, MST I, and Young’s modulus. The highest-achieved compressive strength predicated in strawcrete was 3.8 MPa for MST II, whereas the developed flexural strength (0.69 MPa) was slightly lower than that of hempcrete (0.832 MPa).

Table 4. Developed strawcrete versus hempcrete and Arbolit mechanical property comparison

<table>
<thead>
<tr>
<th>Material</th>
<th>Mixing Technique</th>
<th>Density, kg/m³</th>
<th>Young’s modulus, MPa</th>
<th>Compressive strength, MPa</th>
<th>Flexural strength, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hempcrete [25]</td>
<td>Projection process,</td>
<td>291</td>
<td></td>
<td>0.18</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>based on MST-I</td>
<td>389</td>
<td></td>
<td>0.425</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>434</td>
<td></td>
<td>0.48</td>
<td>0.832</td>
</tr>
<tr>
<td>Developed Strawcrete</td>
<td>MST-I</td>
<td>645</td>
<td>7.1</td>
<td>1.8</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>MST-II</td>
<td>734</td>
<td>10</td>
<td>3.8</td>
<td>0.69</td>
</tr>
</tbody>
</table>
The best weight reduction assigned in hempcrete has a density of less than 1, and the developed strawcrete density varies between 645 and 734 kg/m$^3$ for the different mixing techniques. However, the assigned densities could be enhanced in consideration of the strawcrete classified as a lightweight construction material.

6. Conclusions

The experimental test results of the developed bio-composite matrix made of straw fiber as a replacement to hemp shives show that strawcrete is a renewable sustainable lightweight construction material and a proper alternative to hempcrete. Depending on these results, the elementary related conclusions were as follows:

- MST-II was assigned as the most efficient proper technique for the same loading orientation (normal to tamping layers). The compression strength of specimens prepared via MST-II was 3.8 MPa (for SNT2) against 1.8 MPa (for SNT1), which was prepared by MST-I.
- The normal loading mode was assigned as excellent mode to utilize the full strength capacity of developed matrix. For the same sequence technique, the compression strength dropped from 3.8 MPa to 0.9 MPa as the loading mode changed from normal to parallel one.
- For various mixing techniques or fiber orientations, high-strength specimens exhibited a softening behavior assigned by the modulus of elasticity dropping, whereas the specimen’s compressibility indicated by Poisson ratios confirmed that fiber orientation mode 1 as the matrix of high compressibility.
- The assigned negative Poisson ratio could be affected by the large out-of-plane buckled layers within the matrix of tamping layers orientation 2. The modulus of elasticity increased as the Poisson ratio decreased and resulted in extreme anisotropy.
- For any construction materials, the aspect ratio of dimension related to tested specimens affected their compression strength. For loading mode 1, the compressive strength decreased from 3.8 MPa to 2.807 MPa as the aspect ratio doubled.
- Two different failure modes were assigned for the compressive failure trend. The first related to the first loading mode was distinguished by developing a crushing layer across the section, whereas the specimens tested by the second loading mode failed as buckled bands of discrete plates. The specimens with aspect ratio (2) assigned buckling trend were clearly observed. Two large movement, flexural failure mode distinguished by the main crack continued until failure with the absence of any secondary cracks, which were confirmed by the brittleness challenge of the developed matrix.
- Analysis of the developed strawcrete versus the hempcrete confirmed the high similarity of elasticity response of strawcrete with that of customary hempcrete. Strawcrete showed positive improvement of enhanced compressive strength, which was extended to 1.8 MPa for MST-I against 0.18 MPa for the same mixing technique.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


2597
Investigating Role of Vegetation in Protection of Houses during Floods

Amina a, Ghufran Ahmad Pasha a, Usman Ghani a, Afzal Ahmed a, Fakhar Muhammad Abbas a

*Department of Civil Engineering, University of Engineering and Technology Taxila, Pakistan.

Received 29 May 2019; Accepted 21 September 2019

Abstract

Flood flows have the potential to cause substantial damage to infrastructure, mankind, livestock and agricultural land which all stacks up to greatly affect the financial condition of the region. During 2010 Pakistan floods, more than two million houses were damaged partly or totally [1]. To minimize these types of destructions, inland vegetation can be considered a natural barrier to dissipate the energy of flood flow and limits widespread inundation. This study involves volume of fluid (VOF) modelling approach to figure out the role of vegetation of finite width in energy reduction of flood flow, in front of houses, against: vegetation of varying Aspect Ratio (A/R width-length ratio) and distance between vegetation & houses (Lr). Channel domain was built in ANSYS workbench toolkit and meshing was done in meshing building toolkit. For the postprocessing and simulation, FLUENT was used. Various contour plots & profiles of cross-stream-wise velocities and water level measurements are presented in this paper. The simulation results of cross-stream-wise velocities and water level measurements were identical with experimental data. At vegetation upstream and downstream, velocity reduction observed in higher A/R (2.40) compared to vegetation of A/R-1. Whereas, outside the vegetation and near the walls of channel domain flow velocities were high. The water level was raised on the upstream side of the vegetation due to resistance offered by vegetation. On the upstream side of vegetation, the rise in backwater depth increased by increasing A/R. Contrarily, on the downstream side of vegetation, an undular hydraulic jump was observed in between vegetation and a house. By increasing A/R, the energy loss increases under constant vegetation conditions (Gld = 0.24, Fr0 = 0.70; G = spacing of each cylinder in cross-stream direction and d= diameter of cylinder and Fr0 = initial Froude number) and increase in house distance from IW to 2W, the energy reduction increased from 2.40% to 3.15% which was further increased to 5.04% for another 5W increase in house distance, where W is the vegetation width. Simulation results also shown that with increasing Froude no from 0.60 to 0.70 water level depth has also an incremental pattern which ultimately results in increase in energy dissipation along the varying building distance (IW, 2W & 5W). Thus, to minimize the structural damage, a structure must be located at a safe distance away from the vegetation where flow becomes sub-critical.

Keywords: VOF Modeling; Vegetation; Flood; k-ε Turbulence Model; Aspect Ratio.

1. Introduction

Flood is an enormous amount of water that overflow beyond its normal limits. The impact of floods can cause huge destruction include infrastructure, property and human life. Flood is the increased quantity of fluid (generally water) that inundates the land which is normally dry all year round, from a nearby waterway. Most common inland flood incidents occur near a river or a stream with source of water being a heavy rainfall over a catchment area, either a dam or a levee rupture, or quickly melting snow caps at the mountainous regions. Floods are also caused by tsunamis or hurricanes and that are known as coastal floods as these only do affect the coastal areas and do not travel much distance onto the land. Some notable ones across the world are briefly discussed here. The rise in backwater is increased by both
vegetation density and thickness on upstream side [2]. Vegetations are not only important but multiple factors are important for catastrophic changes during floods [3]. The effect of coastal Vegetation on tsunami run-up heights was investigated and concluded that run up height was reduced by 45% when trees were placed in dense rectilinear arrangement and close to still water level [4]. Critical breaking conditions were used by considering Japanese coastal pine trees. It was observed that inland Vegetations were very helpful to trap much more debris including car debris and large debris [5]. Trees of large diameter can be used to reduce secondary damages caused by driftwood and fluid force intensity [6]. The computational study was given to investigate the flow properties and characteristics. It was observed that velocity was minimum behind the vegetations while in vegetated regions the turbulence was larger, and it was smaller away from the vegetated patch [7].

The analytical model was used to predict the discharge and velocity distribution in compound channel [8]. The flood risk assessment was studied by focusing on frequency analysis (FFA). It was observed that river overflow is more cause of violent variation in flood frequency of downstream areas than dam operations [9]. Non-structural strategies for future flood mitigation in Dhaka city was investigated and concluded that a well-coordinated and balance combination of both structural and nonstructural measures are indispensable for city safety [10]. Vegetation model was used to explain the drag, turbulence and diffusion to unleash the underlying physics, natural range of vegetation density and stem Reynolds’ number [11]. The variation of the vegetative roughness coefficient with the depth of flow was studied and it was found that roughness coefficient reduces with increasing depth under the unsubmerged condition and the vegetation roughness tends to increase at low depths under fully submerged condition [12]. Impacts of coastal vegetation on tsunami reduction was studied and observed that when aspect ratio is 1:4, collision effect behind the Vegetation was maximum. While the aspect ratio is fixed, and the vegetation thickness increased, the impact of a collision of the tsunami behind Vegetation becomes more and as result the fluid force increases [13]. Turbulent flow hydraulic properties were studied in an open channel by using three-layer analytical model through suspended vegetation [14]. The drag force is directly proportional to the velocity of flow and mean difference in relationship was due to nature of vegetation [15]. Turbulence was maximum and by increasing depth flow velocity increases [16]. To control the flood flows vegetation can be used. Crops of maize wheat and barley were used as strip cropping pattern [17].

Investigation of damage to mangroves caused by 2004 Indian ocean tsunami in Thailand. It was found that approximately 70% of the mangrove Vegetation was destroyed by tsunami [18]. Experimental work was performed and observed that two rows Vegetation device more driftwood even in sparse case than the single row Vegetation. When aspect ratio is increased it results in deiving more driftwood [19].Numerical model was presented by considering the tsunami events of 1975 and 1755 of Portugal and resulted tsunami evacuation maps consisting of safe and quick route, located on high ground to save community [20]. Laboratory experiments were conducted to study the overtopping and its effects on height of inundation behind the structure [22]. Spatio-temporal slip model was used and found that damages of tsunami is due to various inundation features including flow velocity, flow depth [23]. Failure of buildings were due to fluid force and tsunami debris and catastrophic model was proposed to predict accurate damages due to debris. Spatio-temporal model was used to study economical loses to building due to tsunami in Omaha beach New Zealand and concluded that the economic loses to buildings are affected by design and number of buildings [24].Frictional factor in water channel during case of sparse grass was less as compared to dense grass. Dense grass caused more resistance to flow [25]. A study was presented to get more accurate velocity profile using three-dimensional model.it was observed that In case of emergent vegetation velocity was uniform except near the bed due to friction it rapidly increases [26].Flow resistance caused by vegetation was observed and concluded that friction loss in branchy trees is 42 percent greater than the leafless trees and when depth increases it gives more accurate results [27]. Velocity decreases when water flows through vegetation and it was high when it was measured prior to vegetation and this decrease becomes less when we move towards downstream side. Velocity dissipation was dominant in case of flexible vegetation [28]. With increasing depth up to 100 cm forces on the building increases after that they are decreasing because irregularity in flow decreases. When angle on inclination increases, load starts [29]. Performance of different countermeasures for tsunami in Sendai city was investigated and concluded that the combination of greenbelt, seawall and elevated highway and road can protect city [30]. The mean streamwise velocity remain constant inside the larger and shorter vegetation but rises at the top of the vegetation. Also, higher velocities were observed in the vegetation patch regions as compared to the gap regions [31].

In Pakistan, the flood disaster mitigation practices are conventional (i.e. construction of dams, barrages and bunds) and these mostly fail during when a major flood occurs. In rest of the world however, the use of inland vegetation has been introduced as an alternative which is both ecofriendly and does not involve as much capital as required for construction and maintenance of hydraulic structures. The idea arose after looking into Japan’s research and their innovations for protection against the tsunami events that occur there in every decade or so. The proposed research is engaged to numerically anticipate the flow behavior in an open channel through emergent vegetation by utilizing k-ε turbulence model. A VOF multiphase method is used in an open channel to predict the velocity distribution and water level measurements in a steady sub critical flow. Also the phenomena of energy loss due to vegetation has been reported as well.
2. Martials and Methods

2.1. Experimental Conditions

2.1.1. Flume Characteristics and Flow Conditions

The experimental data of Pasha and Tanaka (2016) research was used to validate the numerical model [32]. Laboratory experiments were conducted in channel having length 5m, 0.7m in depth and 0.5m in height in Saitama University Japan. Initially the Froude number was initially set 0.7 without vegetation set in a channel and at the channel start the depth of water was set around 4.5cm. The vegetation in staggered arrangement were installed on the channel bed and allocated about 0.75m distance of the channel domain from the from the channel start. Vegetation was displayed as wooden rods with diameter of 0.004m. Using particle image velocimetry (PIV) (Laser Light Sheet: G200, high speed digital CCD camera: K-II, fps: 50–1000, flow analyzing software: FlowExpert2D2C, Katokoken Co., Ltd.) in a cross-wise direction, the velocity distribution at 80% water depth in front of the forest was measured. The flow conditions of the experiment are summarized in Figures 1(a) and 1(b) and Table 1.

Table 1. Experimental Conditions (32) where (A/R) is aspect ratio, (G/d= spacing of each cylinder in cross-stream direction and d= diameter of cylinder) is vegetation density, (Fr_o) is initial Froude number

<table>
<thead>
<tr>
<th>Vegetation A/R</th>
<th>Vegetation Density (cylinders/cm²)</th>
<th>G/D</th>
<th>L/D</th>
<th>Vegetation Type</th>
<th>Depth (m)</th>
<th>Velocity (m/s)</th>
<th>Cylinder arrangements</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.70</td>
<td>0.24</td>
<td>2.1</td>
<td>5.25</td>
<td>Sparse</td>
<td>0.70</td>
<td>0.45</td>
<td>0.465</td>
</tr>
<tr>
<td>2.40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ANSYS workbench toolkit is used to build the experimental setup. Multizone meshing was done in mesh building toolkit using hexagonal elements. The nodes and elements were different due to different aspect ratio in lateral and vertical directions. The relevance center was used as 20% and 50% according to condition. Mesh independence test was also performed to check independency of grids. Simulation and post-processing were carried out in FLUENT. The channel domain considered of air and water, so volume of fluid model was adopted in postprocessing. The boundary conditions for water inlet and outlet were assigned as velocity inlet and pressure outlet respectively. Side walls, bed, and cylinders of channel domain were considered as no-slip wall to manage the solid walls. The solver type and velocity formulation were chosen as pressure-based and absolute.

Figure 1. (a) Experimental model (b) staggered arrangements of vegetation model, where L & G shows the space between each cylinder in direct-stream and cross-stream direction respectively. d is diameter of cylinder, and Wx is vegetation length Wy is vegetation width

2.2. Governing Equations

Continuity equation, momentum equation and k-ε turbulence equations are the governing equations to represent flow of open channels. For steady incompressible flow these equations are written as:

2.2.1. Continuity equation:

The instantaneous continuity equation for an incompressible, homogeneous and steady flow is:

\[
\frac{d u_i}{d x_i} = 0
\]  

(1)

2.2.2. Momentum Equation

The Naiver-Stokes equation for an incompressible, homogeneous flow is given as:
\[
\frac{du_i}{dt} + u_j \frac{du_i}{dx_j} = -\frac{1}{\rho} \frac{\partial p}{\partial x_i} + g_i + \nu \nabla^2 u_i
\]  

(2)

Where \( t \) is time, \( p \) is instantaneous pressure, \( g_i \) is component in the \( i \)th direction of the gravitational acceleration and \( \nu \) is fluid kinematic viscosity.

### 2.2.3. k-\( \varepsilon \) turbulence model

k-\( \varepsilon \) turbulence model was used for turbulence modeling. The turbulence kinetic energy \( k \) and its dissipation rate \( \varepsilon \) obtained from the following equations.

For turbulence kinetic energy \( k \):

\[
\frac{\partial (\rho k)}{\partial t} + \frac{\partial (\rho k u_i)}{\partial x_i} = \frac{\partial}{\partial x_i} \left[ \mu_t \frac{\partial k}{\partial x_i} \right] + 2\mu_t \varepsilon E_{ij} - \rho \varepsilon
\]  

(3)

For turbulent Dissipation rate \( \varepsilon \):

\[
\frac{\partial (\rho \varepsilon)}{\partial t} + \frac{\partial (\rho \varepsilon u_i)}{\partial x_i} = \frac{\partial}{\partial x_i} \left[ \frac{\mu_t \varepsilon}{\sigma_\varepsilon} \frac{\partial \varepsilon}{\partial x_i} \right] + C_{1\varepsilon} \frac{\varepsilon}{k} 2\mu_t E_{ij} E_{ij} - C_{2\varepsilon} \frac{\varepsilon^2}{k}
\]  

(4)

Where

\[
\mu_t = \rho \mu \frac{k^2}{\varepsilon}
\]  

(5)

Where \( C_{1\mu} \) is a dimensionless constant. The Equations 3, 4 and 5 comprise of five constants \( C_{1\mu}, \sigma_k, \sigma_\varepsilon, C_{1\varepsilon} \) and \( C_{2\varepsilon} \). The k-\( \varepsilon \) model utilizes values of these constants as:

\[
C_{1\mu} = 0.09, \sigma_k = 1.00, \sigma_\varepsilon = 1.30, C_{1\varepsilon} = 1.44, C_{2\varepsilon} = 1.92.
\]  

(6)

Diffusivities of \( k \) and \( \varepsilon \) are linked by Prandtl numbers \( \sigma_k \) and \( \sigma_\varepsilon \) to the eddy viscosity \( \mu_t \). To measure pressure term of the exact \( k \)-equation, constants \( C_{1\varepsilon} \) and \( C_{2\varepsilon} \) are used for the correct proportionality between the terms in the \( k \)- and \( \varepsilon \)-equations.

### 2.3. Computational Details of Numerical Model

The computational model was built 2m long, 0.7m in width and 0.1m in height. In staggered arrangement the circular cylinders were installed at the middle of the water stream along the width and 0.7m from the upstream side. The diameter of cylinder was taken as 0.004m. The spacing between the circular cylinders was very fine to get good precision and results. Initial average velocity (0.465m/s), water depth (0.045m), height of cylinders (0.1m) are typical hydraulic conditions used for the modeled domain. The \( k \) & \( \varepsilon \) turbulence specification method was used into the Fluent code, where the turbulence kinetic energy \( k \) and turbulence dissipation rate \( \varepsilon \) was set to 0.0001220(m^2/s^3) and 0.00062741(m^2/s^3). The multi-phase model was used because the channel domain consists of two fluids i.e. water and inlet. The simple scheme & second order upwind discretization scheme were used for pressure-velocity coupling method and VOF multiphase model respectively.

<table>
<thead>
<tr>
<th>Vegetation ((A/R))</th>
<th>Vegetation Density</th>
<th>(G/D)</th>
<th>(L/D)</th>
<th>Vegetation type</th>
<th>(Fr_o)</th>
<th>Water depth ((y))</th>
<th>velocity ((m/s))</th>
<th>Cylindrical arrangement</th>
<th>TKE ((K)) (m^2/s^2)</th>
<th>TDR ((\varepsilon)) (m^2/s^2)</th>
<th>(Lr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.7</td>
<td>0.465</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.7</td>
<td></td>
<td>0.24</td>
<td>2.1</td>
<td>5.25</td>
<td></td>
<td></td>
<td>0.45 m</td>
<td>0.465</td>
<td></td>
<td>0.000122</td>
<td>0.000627</td>
</tr>
<tr>
<td>2.4</td>
<td></td>
<td></td>
<td></td>
<td>sparse</td>
<td>0.65</td>
<td></td>
<td>0.43</td>
<td></td>
<td>0.000166</td>
<td>0.000627</td>
<td>1W,2W,5W</td>
</tr>
<tr>
<td>0.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.39</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3. Results and Discussion

3.1. Validation of Numerical Model

3.1.1. Velocity Profiles

At the upstream side of vegetation, the effect of A/R on flow velocity is discussed in this section. Different color strips are used to represent the pointed positions of flow velocity as shown in Figure 3. The vegetation is allocated at the center of the channel domain due to which pointed positions were organized on only single side of the Center Line (CL) and conditions are symmetrical to CL. Mean velocity of 1cm strip is represented by Figure 3 for the A/R 1, 1.7, 2.4 sparse condition.
<table>
<thead>
<tr>
<th>Strip</th>
<th>A/R</th>
<th>A/R</th>
<th>A/R</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1)</td>
<td>(1.70)</td>
<td>(2.40)</td>
</tr>
<tr>
<td>Strip-1</td>
<td><img src="https://example.com/graph1.png" alt="Graph" /></td>
<td><img src="https://example.com/graph2.png" alt="Graph" /></td>
<td><img src="https://example.com/graph3.png" alt="Graph" /></td>
</tr>
<tr>
<td>Strip-2</td>
<td><img src="https://example.com/graph4.png" alt="Graph" /></td>
<td><img src="https://example.com/graph5.png" alt="Graph" /></td>
<td><img src="https://example.com/graph6.png" alt="Graph" /></td>
</tr>
<tr>
<td>Strip-3</td>
<td><img src="https://example.com/graph7.png" alt="Graph" /></td>
<td><img src="https://example.com/graph8.png" alt="Graph" /></td>
<td><img src="https://example.com/graph9.png" alt="Graph" /></td>
</tr>
<tr>
<td>Strip-4</td>
<td><img src="https://example.com/graph10.png" alt="Graph" /></td>
<td><img src="https://example.com/graph11.png" alt="Graph" /></td>
<td><img src="https://example.com/graph12.png" alt="Graph" /></td>
</tr>
</tbody>
</table>

Distance from Centerline (cm)
(a) ![Graph](https://example.com/graph1.png)
(b) ![Graph](https://example.com/graph2.png)
(c) ![Graph](https://example.com/graph3.png)

Figure 4. Comparison of experimental and numerical mean cross-stream wise velocities for sparse case (a) A/R 1, (b) A/R 1.70, (c) A/R 2.40 the x-axis represents mean velocity with and without vegetation and y-axis represents distance from center line of vegetation, the vertical dashed line represents Vegetation edge.

### 3.1.2. Water Level Measurements

The measurement of water level was computed at five dissimilar pointed positions on the upstream side of the vegetation, which are designated by solid black points at the front face of vegetation, and by red points along the length of channel as denoted in Figure 5.
Increasing the A/R of a vegetation the water level was raised to maximum level at the upstream side of vegetation and water surface incline inside the vegetation was also increased. The rise in water level on the upstream side of vegetation mainly depend upon vegetation conditions. The main cause of rise in water level due to resistance offered by vegetation in the path of water outflow. Both the vegetation density and aspect ratio greatly affect the vegetation because a finite length vegetation was used for this case study. The water depth was not consistent throughout the channel therefore the output location was measured at the front of vegetation marked by black circles at five different locations and along the length of channel domain at every 20cm intervals marked by red circles. In Figure 5, assigned as vegetation center, vegetation end, in between the center and end of the vegetation, and 5cm, 10cm away from the vegetation.

Figure 6 represents the water level distribution for sparse case A/R (1.0, 1.70, 2.40), vegetation density 0.24 cylinders/cm². The X-axis shows the distance taken from the channel start and Y-axis shows water level. It is also shown in the Figure 6 that when the vegetation model was mounted in the channel, it results a certain increase of water level at the front of vegetation than outside the vegetation and consequently there is significant rise in water surface slope inside the vegetation. As the A/R is increased (keeping the vegetation conditions constant), the water level was raised at the front of vegetation. This rise in the water level depresses the velocity of water flow positively at the front face of the vegetation. Fig 6 also shows that water depth away from the vegetation becomes more as compared to without vegetation.

The water profile in the Figure 6 shows that the simulated results are in better agreement with experimental results. This demonstrated the validity of the numerical model.
3.2. Flow Characteristics

3.2.1. Velocity Distribution (Upstream and Downstream Location)

The measured positions were figured out only on single side from the central line (CL) for the reason of symmetrical conditions applies to CL as shown in Figure 7. The vegetation model is mounted at the middle of the water channel domain. Therefore, different typical locations were selected to examine the flow properties. The x-axis represents average velocity with vegetation and without vegetation and y-axis represents distance from center line of the vegetation. The vertical dashed line represents vegetation edge in all cases of A/R (1, 1.70, 2.40), flow velocity in cross stream-wise direction is initiated to decrease in Strip-1 and it maintained the decreased pattern till approaches to the vegetation. Although the flow was steady, still ripples were observed at the forward-facing of vegetation due to back water rise effect of water. The decrease in the flow velocity is because of the energy reduction inside vegetation area. Increase in A/R results a rise in reduction pattern of flow velocity. The larger velocities were observed in Strip-1 because the flow was obstructed to some extent only, whereas the medium velocities were observed in strip (2, 3). The velocity observed higher fluctuations directly upstream of the vegetation at strip 4. (32) have also find the same velocity profiles directly upstream of the vegetation through emergent vegetation. If aspect ratio is increased from A/R 1 to A/R 2.40 with constant vegetation conditions the flow velocity is decreased because vegetation offered more resistance, there. The velocity magnitude in cross stream-wise direction for all cases of A/R (1.0, 1.7, 2.40) directly on the upstream side & downstream side of vegetation are presented in Figure 8.

![Figure 7](image-url)

Figure 7. (a) Area of velocity measurement on the upstream of vegetation (U1, U2, U3, U4) and, (b) downstream vegetation (D1, D2, D3, D4) at five different locations denoted by Different colored strips

<table>
<thead>
<tr>
<th>Location</th>
<th>A/R (1)</th>
<th>A/R (1.70)</th>
<th>A/R (2.40)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Upstream</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 cm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 cm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Downstream</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 cm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 cm</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

![Figure 8](image-url)

Figure 8. Mean cross-stream wise velocities, sparse vegetation case, vegetation density 0.24 cylinders/cm² (a) A/R =1 (b) A/R 1.70 (c) A/R 2.40 upstream location and downstream location
3.3. Water Profiles

These are the water depth profile along the length of the channel measured at the center of channel domain against three different Froude number (0.70, 0.65, 0.60) with constant vegetation conditions. The house models were placed at three different distances (1W, 2W, 5W) downstream of the vegetation. These curves follow the scheme of flow shown in Figure 9.

Figure 9 shows clearly shows the backwater rise, undulations in water depth after the vegetation which possibly depicts a undular hydraulic jump and the difference caused to water depth with addition of house at the downstream side of vegetation. The resulting plots show that as distance increases, the difference in depth before and after the vegetation can be seen increasing representing increased energy dissipation. The presence of the building downstream results in increased water depth between the vegetation and building.

<table>
<thead>
<tr>
<th>Fr</th>
<th>A/R (2.40)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.70</td>
<td><img src="" alt="Water depth profile" /></td>
</tr>
<tr>
<td>0.65</td>
<td><img src="" alt="Water depth profile" /></td>
</tr>
<tr>
<td>0.60</td>
<td><img src="" alt="Water depth profile" /></td>
</tr>
</tbody>
</table>

![Figure 9. Water level measurement of channel domain (sparse Vegetation), vegetation density: 0.24 Cylinders/cm²](attachment:figure9.png)

**3.4. Contours of Velocity Distribution**

The contours of the velocity on the transverse planes for all aspect ratios are captured by the numerical mode as shown in Figures 10 and 11. It can be seen from the Figures 10 and 11 that near the vegetation zone the velocities were reduced. The reduced velocity is because of the vegetation obstruction, because the depth of water was raised directly on the upstream of vegetation. Significant effects of the presence of vegetation was observed on distribution of velocities...
within vegetation patch zones, upstream side of vegetation, in the gap region, and near the wall of channel. Maximum amount of velocities was observed in the gap region and near the bed. In all cases of sparse vegetation, vegetation density (0.24 cylinders/cm²), A/R (1, 1.7, 2.4), the velocity is started to decrease in strip-1 and follow this reduction pattern till approaches the vegetation.

In A/R 1.0 Strip-1 (6-7 cm) the velocity is started to decrease directly upstream of the vegetation and maximum velocity in the gap region and near the walls of channel. In strip 2 (2-3 cm), strip 3 (3-4 cm) the magnitude of velocity is further reduced because this region is near to the vegetation. The legend shows the magnitude of velocity. Similarly, the same behavior is shown in A/R 1.7 and 2.4. Hence the contours clearly show that the velocity is reduced in higher A/R (2.40) as compared to lesser aspect ratio A/R (1).

By comparing the contours of upstream and downstream sides of the cross stream-wise velocities for sparse case, vegetation density (0.24 cylinders/cm²) and A/R (1, 1.70, 2.40), the lower velocities were observed directly at the downstream side of vegetation in comparison to upstream side of vegetation, due to resistance offered by vegetation elements. Average velocities were observed at the middle of channel and maximum velocities were observed away from the vegetation patch.

<table>
<thead>
<tr>
<th>A/R (1)</th>
<th>A/R (1.70)</th>
<th>A/R (2.40)</th>
</tr>
</thead>
<tbody>
<tr>
<td>U1</td>
<td>U2</td>
<td>U3</td>
</tr>
<tr>
<td>Z (m)</td>
<td>Z (m)</td>
<td>Z (m)</td>
</tr>
<tr>
<td>Y (m)</td>
<td>Y (m)</td>
<td>Y (m)</td>
</tr>
</tbody>
</table>

Figure 10. Contour (upstream location) plots of mean cross stream-wise velocities along the vertical transverse surfaces of all the cases of sparse vegetation, vegetation density 0.24 cylinders/cm² (a) A/R= 1.0, (b) A/R =1.70, (c) A/R =2.40. The dashed portion show the vegetation patch area
Table 1. Contour (Downstream location) plots of mean cross stream-wise velocities along the vertical transverse surfaces of all the cases of sparse vegetation, vegetation density 0.24 cylinders/cm². A/R =1.0 (a) A/R =1.70 (b) A/R =2.40 (c).

Figure 11. Contours obtained at different house distance from vegetation. These plots show that how house distance between vegetation and house model affects in lowering down velocity profiles at downstream side during flood flow. Simulation was carried out at three different house distance: \(1W\), \(2W\), & \(5W\), while keeping A/R =2.40. It is clear from the contour plot that velocities are reduced with increasing house distance. So, to get housing safe from flood flow they should be at safe distance from vegetation where flow becomes sub-critical. The reduced velocity profiles along increased house distance results in increased energy dissipation which ultimately make housing safer from flood flow.

Figure 12. Contour (Front face of house at section \(y=0.9216\) m for \(1W\), \(y=1.0124\) m for \(2W\), \(y=1.2848\) m for \(5W\)) plots of mean cross stream-wise velocities along the vertical transverse surfaces of all the cases of sparse vegetation, vegetation density 0.24 cylinders/cm², A/R =2.40. The dashed portion show the vegetation patch area

3.5. Velocity Distribution at the Front of House Model

Figure 12 shows contours obtained at different house distance from vegetation. These plots show that how house distance between vegetation and house model affects in lowering down velocity profiles at downstream side during flood flow. Simulation was carried out at three different house distance: \(1W\), \(2W\), & \(5W\), while keeping A/R =2.40. It is clear from the contour plot that velocities are reduced with increasing house distance. So, to get housing safe from flood flow they should be at safe distance from vegetation where flow becomes sub-critical. The reduced velocity profiles along increased house distance results in increased energy dissipation which ultimately make housing safer from flood flow.
3.6. Energy Dissipation

Relationship between specific energy $E$, water depth $y$ and velocity $V$ can be shown as [33]:

$$E = y + \frac{\alpha V^2}{g}$$

(7)

Where $\alpha$ is velocity variation coefficient and $g$ is gravitational constant. In the current research work $\alpha$ is taken as 1 (2). This equation can be used to find energy dissipation, which is difference between specific energy at the upstream side and downstream side of vegetation. In the present research work, we find Energy dissipation in terms of total energy loss ($\Delta E = E_1 - E_2$) and relative total energy loss ($\Delta E/E$).

3.6.1. Total Relative Energy Loss with Aspect Ratio

Velocity profile and water depth with respect to $A/R$ has been shown in Figures 6 and 8. It is clear from Figure 6 that water depth is increasing with increasing $A/R$ on the front side of vegetation due to back water rise, also slope of water surface in the vegetation zone increases. As water flow becomes critical on the downstream side through mounted vegetation, a low intensity profile hydraulic jump is created on the downstream side (Figure 13 Flow structure scheme), which ultimately support Energy Loss. Similarly, velocity profile in Figure 8 shows velocity reduction at the downstream side of vegetation than on the upstream side of vegetation, resulting a contribution to Energy Loss.

Extracting data from Figures 6 and 8, a plot of total relative energy loss with respect to $A/R$ has been drawn. The $A/R$ of vegetation have significant effects on the downstream flow. Increase in the aspect ratio of vegetation causes to reduces the velocity of water at the downstream side of the vegetation and ultimately results in energy loss. Figure 14 shows variation of total energy loss ($\Delta E/E$) with respect to aspect ratio. The relative total energy loss is greatest for sparse case with $A/R = 2.40$ and least for $A/R = 1$ (i.e. 6.2 and 2.04%) by keeping vegetation density (0.24 cylinders/cm$^2$) and initial Froude number 0.70 constant.

![Figure 13. Flow structure scheme and definition of different parameter](image1)

![Figure 14. Relationship between relative total energy loss ($\Delta E/E$) and $A/R$ (1, 1.70, 2.40)](image2)
3.6.2. Total Relative Energy Loss with Froude Number

Also Figure 16 shows the effect of Froude no. on total relative Energy loss at varying house distance for the single sparse case. It is clear from the Figure 9 that with increasing Froude no, water level depth has also incremental pattern. This increment in water depth ultimately results in increasing energy loss. In Figure 16 curve is drawn for the vegetation model with characteristics: Vegetation Density=0.24 cylinders/cm² & Froude number = (0.7, 0.65, 0.6). For this single case of vegetation and in between three Froude numbers (0.7, 0.65, 0.6) the energy loss increased exponentially. The increase in building distance from 0.9216m (1W) to 1.0224m (2W) increase the relative total energy loss from 2.40 to 3.15% and when increase to 5W (1.2948), the energy reduction further increased 5.04%.

![Figure 15. Flow structure scheme with house model](image1)

![Figure 16. Relationship between relative total energy loss and house distance](image2)

4. Conclusions

In this research work, Velocity distribution, water profiles & Energy dissipation of steady sub critical flow through emergent vegetation have been computed using VOF multi-phase method. A k-ε turbulence model was used in this study. The research can be concluded as follows:

- A numerical study has been done using Ansys software to predict the mean cross stream-wise velocities in an open channel and water level measurements against three different A/R (1, 1.7, 2.40) through emergent vegetation. The simulation results show that multi-zone mesh converges rapidly and have good correspondence between numerical model and experimental results.

- From the contour plots the minimum magnitude of cross stream-wise velocities was found within vegetation zone as compared to the gap regions.

- Velocity profiles and contour plots depicts the reduction in velocities along the upstream side of vegetation as well as in the downstream side of the vegetation. Lower velocities in upstream side were due to back water rise effect while in downstream side it was due to resistance of vegetation elements in the path of flow.

- At the vegetation upstream side, for constant vegetation density (G/d), velocity reduction increases with the increase in A/R (1.0, 1.70, 2.40) along the channel length. Similarly, the A/R of vegetation have significant effects.
on the downstream flow. Increase in the aspect ratio of vegetation causes to reduces the velocity of water at the downstream side of the vegetation

- Water depth also increases with increasing A/R from 0.60 to 0.70 along the channel.
- The total relative energy dissipation by the vegetation is greater for the model of higher aspect ratio. Increase in aspect ratio from 1 to 2.40 resulted in more energy reduction from 2.02 to 6.2%.
- The building distance greatly affects the energy dissipation. For constant vegetation conditions and single sparse case (A/R=2.4), the increase in building distance from 1W (0.9297m) to 2W (1.0205 m), the energy reduction increased from 2.40 to 3.15% which was further increased for distance 5W (1.2948) i.e. 5.04%
- Simulation results shows the effect of Froude no. on total relative Energy loss at varying house distance for the single sparse case. According to results with increasing Froude no from 0.60 to 0.70 water level depth has also an incremental pattern which ultimately results in increase in energy dissipation along the varying building distance (1W, 2W & 5W).

According to the observations, it is recommended that vegetation should be of high aspect ratio for greater energy dissipation and the building should be at a great distance from the vegetation to obtain maximum effect of the vegetation and hence safety.

These results are vital for the optimum vegetation design for the protection of structures. In the future, the aim is to develop a numerical model with different densities of vegetation and other actual ground conditions to find minimum optimum building distance to be observed and followed from the flood plane to reduce flood destructions with vegetation.

**5. Notations**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A/R</td>
<td>Aspect ratio (width-length)</td>
</tr>
<tr>
<td>d</td>
<td>Diameter of cylinder</td>
</tr>
<tr>
<td>G</td>
<td>Spacing of each cylinder in cross stream-wise direction</td>
</tr>
<tr>
<td>Fr0</td>
<td>Initial Froude number</td>
</tr>
<tr>
<td>L</td>
<td>House distance</td>
</tr>
<tr>
<td>W</td>
<td>Width of vegetation</td>
</tr>
<tr>
<td>Wx</td>
<td>Spacing between each cylinder</td>
</tr>
<tr>
<td>Wy</td>
<td>Vegetation length</td>
</tr>
<tr>
<td>TKE</td>
<td>Turbulence kinetic energy</td>
</tr>
<tr>
<td>TDR</td>
<td>Turbulence dissipation rate</td>
</tr>
<tr>
<td>m</td>
<td>Meter</td>
</tr>
<tr>
<td>s</td>
<td>Second</td>
</tr>
<tr>
<td>cm</td>
<td>Centimeter</td>
</tr>
<tr>
<td>Vf</td>
<td>Average velocity with vegetation</td>
</tr>
<tr>
<td>Vo</td>
<td>Average velocity without vegetation</td>
</tr>
<tr>
<td>E</td>
<td>Specific Energy at forest front</td>
</tr>
<tr>
<td>y</td>
<td>Water depth</td>
</tr>
<tr>
<td>V</td>
<td>Velocity of water (ms-1)</td>
</tr>
<tr>
<td>a</td>
<td>Velocity variation constant</td>
</tr>
<tr>
<td>g</td>
<td>Gravitational constant</td>
</tr>
<tr>
<td>ΔE%</td>
<td>Total energy loss</td>
</tr>
<tr>
<td>ΔE/E%</td>
<td>Relative total energy loss</td>
</tr>
<tr>
<td>Δh</td>
<td>water depth with vegetation</td>
</tr>
<tr>
<td>ho</td>
<td>Initial water depth without vegetation</td>
</tr>
<tr>
<td>HGL</td>
<td>Hydraulic Grade Line</td>
</tr>
<tr>
<td>EGL</td>
<td>Energy Grade line</td>
</tr>
<tr>
<td>E1</td>
<td>Specific Energy at forest front</td>
</tr>
<tr>
<td>E2</td>
<td>Specific Energy at a point of minimum water depth where the flow is super-critical</td>
</tr>
</tbody>
</table>

**6. Acknowledgements**

The authors are very thankful to Higher Education Commission (HEC), Pakistan, for the services of Computational Fluid Dynamics (CFD) software at University of Engineering & Technology, Taxila, Pakistan, which was used to perform this research work.

**7. Conflicts of Interest**

The authors declare no conflict of interest.
8. References


Parametric Study of the Modal Behavior of Concrete Gravity Dam by Using Finite Element Method

Seyed Reza Jafari a, Majid Pasbani Khiavi b *

a M.Sc., Faculty of Engineering, University of Mohaghegh Ardabili, Ardabil, Iran.
b Associate Professor, Faculty of Engineering, University of Mohaghegh Ardabili, Ardabil, Iran.

Received 28 June 2019; Accepted 11 October 2019

Abstract

Calculating the natural frequency of dams is an essential part of its seismic behavior analysis. Therefore, it is important to calculate the natural frequency. This paper aims simulation and analysis the finite element (FE) model of the Koyna concrete gravity as a case study. For the investigation of the suitable mesh size to achievement the grid independence, the element size considered as a variable parameter and calculated its optimized value by using the Response Surface Optimization (RSO) method. In the independent grid, the Error Contour utilized for controlling mesh quality, which indicates fast variations of the energy in the adjacent elements and can recognize parts of the model that has a high error in calculating responses. The modal response of the dam with a rigid and flexible foundation with and without mass were appraised. The results indicated that modal frequencies in the condition of with and without Pre-stress were different value in all cases. Moreover, the frequency of first four modes by increasing mass and decreasing the stiffness of foundation, frequencies in the case without initial condition (without Pre-stress) has a slightly increased and in the case with initial condition (Pre-stress) had considerable decrease.

Keywords: Modal Analysis; Concrete Gravity Dam; Grid Independence; Response Surface Optimization (RSO); Error Contour; Pre-stress.

1. Introduction

Calculating the natural frequency of dams is an essential part of its seismic behavior analysis. Results of the frequency analysis can be used to damping calculation and the response spectrum analysis in order to evaluate the seismic behavior of the dam. Therefore, it is important to calculate the natural frequency. Modal analysis is the process of determining the intrinsic dynamic specifications of the system in the form of natural frequencies, damping coefficients, and the shape of modes and applying them to create a mathematical model of the dynamic behavior of the system. Modal analysis is focused on the principle that vibration response of the linear dynamic system is a collection of simple coordinate actions that are in vibration modes. This concept is similar to using Fourier combination of sine and cosine waves to show a complex wave. Vibration mode shape is related to system dynamic which is determined by physical specifications such as mass, stiffness, damping, and their distribution method. Each mode is described based on the modal parameters of the same modes, including the natural frequency, the modal damping coefficient, and the displacement pattern in that mode which is called mode shape. Mode shape may be real or imaginary and each mode corresponds to a natural frequency. The contribution amount of each natural mode in the vibration of the entire system depends on the specifications of the stimulation source and mode shape [1].

*Corresponding author: pasbani@uma.ac.ir

doi: http://dx.doi.org/10.28991/cej-2019-03091437

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
Chopra [2] observed that the response of the short-period vibration structure, such as the concrete dams subjected to seismic was largely influenced by the fundamental mode of vibration, and in his analyzes, also concluded that the vertical components of the ground acceleration had little influence on the structure response. Chopra and Chakrabarti [3] introduced a general procedure for the analysis of the response of concrete dams, including the dynamic effects of water and the flexible foundation to the horizontal and vertical components of soil movement. Fenves and Chopra [4] developed a semi-analytical-numerical technique to analyze the earthquake response of concrete gravity dams using two special cases (a) full reservoir dam supported by a rigid foundation; and (b) an empty reservoir dam supported by a flexible foundation, concluding that in the first case, the dam-reservoir effect and the bottom of the lake are relevant to the dam response, whereas in the second case the dam response is only related to the foundation and the dam. Dominguez et al. [5] studied the effect of the reservoir-foundation interaction wherein they proposed a contour integral technique to investigate the response of the dam reservoir-sediment-foundation systems subject to soil displacements. Arabshahi and Lofti [6] conducted a study on the mechanisms of natural vibration due to damage on the interface at the bottom of the dam, starting from a plasticity-based formulation using the local stresses of the element on the interface to model the sliding as well as the partial cracks along the foundation.

Shariatmadar and Mirhaj [7] evaluated the hydrodynamic pressures induced due to seismic forces and Fluid-Structure Interaction (FSI) using the ANSYS program. The results showed that the accurate modeling of dam-reservoir-foundation and their interaction considerably affects the modal periods, mode shapes and modal hydrodynamic pressure distribution. Pashbani Khiavi et al. [8] assumed that the boundary and the reservoir interface are vertical, the reservoir base is rigid and horizontal, and the fluid in the reservoir is not incompressible and viscous. The relevant boundary conditions and equations have been applied according to the finite element method (FEM) with considering horizontal and vertical earthquake components. Sevim et al. [9] determined the dynamic characteristics of a prototype arch-reservoir-foundation dam system using the modal analysis method including local vibration tests in the arc dam model identifying its natural frequencies and modes of vibration. Khosravi et al. [10], Khosravi and Heydari [11] considered geometry variables to analyze the gravity dams, which is two-dimensional finite element (FE) model included a dam, reservoir, and foundation, by using ANSYS APDL. The results showed that considering the dam-reservoir-foundation interaction has an important role in safely designing a gravity dam.

Mahdizadeh and Ghanbari [12] concerned a new formulation for natural frequency of gravity dams utilizing the analytical method. In this method, shear wave velocity and height of the dam are two parameters which are used for obtaining natural frequency and show that there is no significant difference in the dam with various heights. Aghajanzadeh and Ghaemian [13] investigated the seismic performance of concrete gravity dam to evaluate the effect of foundation on the nonlinear response using the FEM. They used an elasto-plastic formulation to model the foundation. Mohr-Coulomb model and smeared crack model were utilized for the yield of the foundation and dam body, respectively. The results show that cracks form at the crest and hill of the dam. Pashbani Khiavi [14] investigated the reservoir bed characteristics effect on reducing the induced pressure in the reservoir. Results confirmed a high dependence of responses to the reservoir bottom absorption. Additionally, Pashbani Khiavi [15] investigated the influence of the concrete stiffness on the seismic responses of concrete gravity dams by the Monte Carlo simulation. According to the results, the optimized value of the concrete Young Modulus to access the confident response of the structure was achieved, which was important concerning the economic aspects. Seleemah et al. [16] used the ANSYS software to analyze the dynamic response of the dam reservoir-foundation system to showing that the results of stresses and displacements are significantly affected when it has flexibility in the foundation. Taylan, and Aydin [17] considered the Darideresi-II dam and used ANSYS Workbench to investigate the responses under different earthquake accelerations. Silvaer and Pedroso [18] evaluated the influence of the foundation and reservoir on the dynamic response of concrete gravity dams as a function of their parameters in terms of natural frequencies and mode of vibration. The dam reservoir-foundation interaction will be investigated through the modal analysis by the FEM via the ANSYS APDL software.

In this paper, the ANSYS standard version is used for modeling and analysis. It should be noted that the standard version of this software has the ability to apply various boundary conditions and the effects of interaction between the dam, reservoir and foundation. To apply the interaction effect, the FSI command contained in this software has been used. ANSYS is a software program based on the FEM and is used to give more accurate results for complicated geometries. In this method, the geometry is divided into small parts and the boundary conditions are applied. Finally, the results are obtained for each node or element [17]. During the process of this study, ANSYS Workbench software is used [19]. The main innovations of this paper are the Response Surface Optimization (RSO) method utilization to investigate the influence of grid dimension on responses and the Error Contour method utilization to verify the appropriate dimension for the minimum computational error on achieving the grid independence.

2. Case Study

In this study for simulation and analysis a non-overflow monolith of the Koyna concrete gravity dam located in India [20] with 103 m height, two-dimensional, and the plane stress behavior as illustrated in Figure 1 has been considered as a case study to evaluate the modal performance of Koyna dam under modal analysis. Due to the
devastating earthquake experience, this dam has been more studied. Moreover, the properties of the allocated material for the concrete dam, the foundation, and the reservoir based on experience and background of studies are listed in Table 1 [21-28].

In the ANSYS Workbench, several analysis systems are linked together to analyze the model. In the Structural Analysis part, the material specifications are assigned and boundary conditions are specified for the dam model. Also in this section, the water has given force to the dam, which is defined as hydrostatic pressure. Then, RSO from Design Exploration linked in Structural Analysis part to the parametric study of the element size to approach the grid independency. Finally, Modal Analysis linked to the Structural Analysis part to determine the modes of the structure. If the model statically analyzed and induced responses are applied as an initial condition for new analysis, this type of analysis called Pre-stress condition and if there is not any initial condition, the analysis is considered as without Pre-stress. For modeling and analysis, Koyna dam with two conditions of the rigid and flexible foundation with and without mass are simulated and modal analyzed. Then free vibration frequencies with and without Pre-stress are extracted and frequency responses of modes are compared and assessed in these six specimens (chart available in Figure 9).

![Figure 1. The geometry and FE model of the Koyna dam [29]](image)

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dam body (Concrete)</td>
<td>Density (Kg/m^3)</td>
<td>2643</td>
</tr>
<tr>
<td></td>
<td>Young’s modulus (MPa)</td>
<td>31027</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>Damping</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>Compressive initial yield stress (MPa)</td>
<td>12.6</td>
</tr>
<tr>
<td></td>
<td>Compressive ultimate stress (MPa)</td>
<td>27.11</td>
</tr>
<tr>
<td>Foundation (Bedrock)</td>
<td>Density (Kg/m^3)</td>
<td>2701</td>
</tr>
<tr>
<td></td>
<td>Young's modulus (MPa)</td>
<td>16860</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio</td>
<td>0.18</td>
</tr>
<tr>
<td>Reservoir (Water)</td>
<td>Density (Kg/m^3)</td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td>Bulk modulus (GPa)</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio</td>
<td>0</td>
</tr>
</tbody>
</table>

### 3. Grid Independence

In this paper, the FEM is used, which is the science of discretization the continuous system into small and separate elements and analyzing discrete elements. If the type and dimension of the elements are suitably appointed, the simulated model has exact results and close behavior to the behavior of the real model. Therefore, there is less error in the calculations [30]. Though the fine mesh or element size reduces the error and augment the accuracy of the response, but should insomuch be enough shrinking that by changing the element size, response changes in the model should have
negligible. If changing the grid dimension does not affect the model responses, and the responses are independent of the grid dimension, namely reached the response stability. This concept of grid discretization to attain the convergence and stability of the response from the mesh or element size is called grid independence. To discretization the FE model, the 8-node square element has been used and the mesh independency studied. For this cause, the element size in the RSO parametric analysis method between 0.1 to 2 m has been chosen as input variable parameter and the resultant displacement of the crest, Pre-stress frequency of the first mode, and the number of elements and nodes for dam model were considered as the output variables.

![Figure 2. The curve of the element size variation effect on the dam crest displacement](image)

![Figure 3. The curve of the element size variation effect on the frequency of the first mode](image)

![Figure 4. The curve of the element size and node number variation effect on the dam crest displacement](image)
Figures 2 to 6 shows the effect of the element size and number on the responses for the achievement of grid independence. Figure 2 and 3, indicates the effect of the element size between 0.1 m to 2 m on the consequent of the displacement of dam crest and frequency of the first mode respectively, which by decreasing the size of an element, displacement and frequency achieved convergence. Figure 4 and 5, illustrates the effect of the element size and node number gave out from the grid discretization on the dam crest displacement and frequency of the first mode. It is obvious from figures by decreasing the element size, the number of element and node will be increased and displacement reaches convergence. Figure 6 presents the effect of the element size variation upon the number of element and node to attain the grid discretization, which by decreasing the element size, the number of element and node will be increased. When the element size becomes excessively small, the number of element and node suddenly increases and reaches to divergence and the calculation time will be time-consuming.

By surveying Figure 2 to 5, it can be observed that with the certain element size in which the displacement curve and the frequency of the first mode tend to convergence, the number of elements and nodes in Figure 6 get into divergence threshold. As the dimensions of the grid diminish, the model’s stiffness decreases and becomes closer to the actual model. Because the frequency has a direct relation with the stiffness, in Figure 3 and 5, it is obvious that the element refinement cause increase the element and node number and reduce the first mode frequency (in the Pre-stress condition) and stiffness. Furthermore, by increasing the number of element and node, the dimension of the matrix equation become larger and solve will be time-consuming. Considering the concept of grid independence, the appropriate element size can be estimated as 0.3 m in which the responses have reached to the convergence and the discretizing of the grid is not required smaller. The specification of the element type and the final discretization is briefly listed in Table 2.
Table 2. Summary specification of the element type used in the discretization

<table>
<thead>
<tr>
<th>Element Type</th>
<th>PLANE 183 2D Quad-8 Node</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of Elements</td>
<td>31132</td>
</tr>
<tr>
<td>No. of Nodes</td>
<td>94293</td>
</tr>
<tr>
<td>Element Size (m)</td>
<td>0.3</td>
</tr>
</tbody>
</table>

In the following, by considering the characteristics given in Table 2, the analysis is carried out and the mesh quality for 0.3 m element size is evaluated. In Figure 7, the distribution of the quality of the elements with three separate criteria is assessed, which has a value between 0 and 1. These three criteria are [19]:

- Skewness Quality: if closer to 0, it has higher mesh quality and lower elongation of the elements.
- Element Quality: if closer to 1, it has a higher mesh quality and square elements.
- Orthogonal Quality: if closer to 1, it has a higher quality with elements of about 90 degrees.

Figure 7. Chart of the mesh generating quality of the dam system with the rigid foundation

Figure 7 presents the distribution of the number of elements in terms of the three criteria of the mesh quality that the coefficient of all three criteria were close to the appropriate value (e.g. Orthogonal Quality has 15400 elements with 0.985 coefficient). With remarking to the range of variations in the number of elements and coefficients in any type of criterion, it can be figured out that almost all square elements have less elongation and close to 90 degrees, which indicates the quality and composition of the appropriate discretization for the FE model.

4. Error Contour

Utilization of the Error Contour is a method for controlling the mesh quality of the FE [19], which indicates the rapid variation of the energy in the adjacent elements. The Error Contour can identify areas of the model that have high error rates in the stresses calculating and at which parts need to make the element smaller to obtain more accurate responses. Therefore, improving the mesh quality cause reduces energy difference in the adjacent elements. After surveying the grid independency, the discretization quality (mesh quality) of the case study model by using the Error Contour has been investigated (in Figure 8). The most critical area is in the heel of the dam, which its energy difference is not significant and indicates the suitability of the element size.
5. Frequency Analysis of the Model

Frequency or modal analysis is a method for determining vibration specifications of the structure. The structure without loading can vibrate in special frequencies. These frequencies are natural frequency and vibrations are free vibrations. The deformation of the structure in each frequency is named mode shape. If the structure is forced to vibrate at a particular frequency, it can vibrate in frequencies other than the natural frequency. This frequency is excitation frequency and the vibration is forced vibration. If the natural frequency is close to excitation frequency, the resonance phenomenon can occur. In other words, if the structure is being forced to vibrate at its natural frequency, resonance will occur and large amplitude vibrations will be observed. Therefore, modal analysis permits the designer to prevent severe vibrations or vibrates in special frequency [19]. Figure 9 shows a chart of analysis process during this paper (responses signed with color and series).

5.1. Frequency Analysis of the Dam with a Rigid Foundation (Model A)

In this section, the dam system with a rigid foundation is considered for evaluation of the mesh independence and natural frequencies. Estimation of the natural frequencies of the dam was determined by considering the conditions of plane stress without interaction between the dam, reservoir, and foundation. The free vibration frequencies with and without the Pre-stress condition (A1 and A2) for the first four modes derived from the simulation of the dam system with the free vibrational frequencies of Huang [21] were given in Table 3.

<table>
<thead>
<tr>
<th>Mode No.</th>
<th>Calculated free vibration (Hz)</th>
<th>Reference free vibration (Hz) [21]</th>
<th>% Error</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without Pre-stress (A1)</td>
<td>With Pre-stress (A2)</td>
<td>With Pre-stress</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>3.0801</td>
<td>3.08</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>8.2309</td>
<td>8.23</td>
</tr>
<tr>
<td>3</td>
<td>1E-05</td>
<td>10.825</td>
<td>10.82</td>
</tr>
<tr>
<td>4</td>
<td>6.1396</td>
<td>15.984</td>
<td>15.98</td>
</tr>
</tbody>
</table>

The frequency of vibrational modes has a significant difference in two cases of frequency analysis. Also, the error rate obtained from comparing the simulated model with the reference model is very low, indicating the accuracy of the model. Figure 10 shows the mode shape for the first four modes of free vibration frequency responses of Model A2. Also, Figure 11 illustrated the cumulative percent of the mass contribution for each mode in the Model A2 upon the vertical, horizontal, and rotational direction around the perpendicular axis. If the superposition principle is used, it is possible to use the first six modes that provided 90% of the amount of mass participation.
5.2. Frequency Analysis of the Dam with a Flexible Foundation without Mass (Model B)

In this section, the dam system with elastic foundation without mass simulated and natural frequencies is calculated. Table 4 shows the comparison of the natural frequency of free vibration for two cases of with and without Pre-stress. Results show the increase of mode frequency in Pre-stress model relative to model without Pre-stress.

The comparison of Table 3 and Table 4 frequencies without Pre-stress has little increase and frequencies with Pre-stress decreases because of the decrease of foundation rigidity.

<table>
<thead>
<tr>
<th>Mode No.</th>
<th>Calculated free vibration (Hz)</th>
<th>Without Pre-stress (B3)</th>
<th>With Pre-stress (B4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.93E-06</td>
<td></td>
<td>2.3916</td>
</tr>
<tr>
<td>2</td>
<td>8.30E-06</td>
<td></td>
<td>5.7819</td>
</tr>
<tr>
<td>3</td>
<td>1.98E-05</td>
<td></td>
<td>6.1909</td>
</tr>
<tr>
<td>4</td>
<td>6.1662</td>
<td></td>
<td>11.166</td>
</tr>
</tbody>
</table>

Figure 12 shows the mode shapes of the first four modes for free vibration of the model with the Pre-stress condition. Also, Figure 13 illustrated the cumulative percent of the mass contribution for each mode in the Model B5 upon the vertical, horizontal, and rotational direction. If the superposition principle is used, it is possible to use the first three modes that provided 90% of the amount of mass participation.
In this section, the dam with a flexible foundation with mass analyzed and the natural frequencies are calculated. Obtained results of the natural frequencies for two cases of with and without Pre-stress (C5 and C6) have been illustrated in Table 5. Comparison of the results shows that the Pre-stress condition caused increasing the frequencies. Also, the comparison of obtained frequencies for two cases of the flexible foundation with and without mass in Table 4 and Table 5 disclose that the model without mass frequencies (C5) has rather than the model with mass (C6).

**Table 5. Free vibration frequencies of the dam system with the elastic foundation with the mass in the first four modes**

<table>
<thead>
<tr>
<th>Mode No.</th>
<th>Calculated free vibration (Hz)</th>
<th>Without Pre-stress (C5)</th>
<th>With Pre-stress (C6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.50E-06</td>
<td>0.1046</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>6.66E-06</td>
<td>0.23846</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.25E-05</td>
<td>0.38332</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0.86139</td>
<td>6.1425</td>
<td></td>
</tr>
</tbody>
</table>
Figure 13 shows the mode shape for the first four modes of the frequency response of free vibration with Pre-stress. Furthermore, Figure 14 shows the cumulative percent of the mass contribution for each mode in the Model C6 upon the vertical, horizontal, and rotational direction. If the superposition principle is used, it is possible to use the first two modes that provided 80% of the amount of mass participation. And a summary of the frequency responses of the six case studies illustrated in Figure 15. It is educed from Figure 15 that in the case of Pre-stress conditions, mass matrix, and stiffness matrix cause reduced the stiffness and frequency.
6. Conclusion

A parametric study of the effect of different conditions on the frequency behavior of the dam was investigated using the FEM. Initially, due to the effect of discretization and dimensions of the elements on the stiffness and softness of the model, effect of the mesh size on frequencies was investigated to optimize the dimensions of mesh based on grid independence. RSO method confirmed that has a proper capability to appraise parametric and sensitive analysis in order to achieve the optimum element size remarked by the grid independence concept. Error Contour method was used to investigate the energy variation of adjacent elements by the optimized element size. The results exposed that the RSO method has an appropriate application to attain minimum energy difference and consequently, the quality of the grid meshing is acceptable and the responses will be accurate.

According to the analyses, appraisal the results revealed that the Pre-stress condition as an initial condition caused increasing the modal frequencies. Moreover, by decreasing in foundation stiffness and increasing the mass, frequencies in case of with Pre-stress decreased and in case of without Pre-stress has more decreased. Therefore, it can be deduced that for accurate analysis of the dam-reservoir.foundation model analysis, it has been necessary to consider the exact boundary condition and initial condition to assess the model with a proper viewpoint.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


An Investigation of the Fundamental Period of Vibration for Moment Resisting Concrete Frames

Ahmed N. Mohamed a, Khaled F. El Kashif b, Hamed M. Salem c

a M.Sc. Student, Structural Engineering Department, Cairo University, Giza, Egypt.
b Assistant Professor, Structural Engineering Department, Cairo University, Giza, Egypt.
c Professor, Structural Engineering Department, Cairo University, Giza, Egypt

Received 24 May 2019; Accepted 30 September 2019

Abstract

The determination of fundamental period of vibration for structures is essential to earthquake design. The current codes provide empirical formulas to estimate the approximated fundamental period and these formulas are dependent on building material, height of structure or number of stories. Such a formulation is excessively conservative and unable to account for other parameters such as: length to width ratios, vertical element size and floors area. This study investigated the fundamental periods of mid-rise reinforced concrete moment resisting frames. A total of 13 moment resisting frames were analyzed by ETABS 15.2.2, for gross and cracked eigenvalue analysis and Extreme Loading for Structures Software® or ELS, for non-linear dynamic analysis. The estimated periods of vibration were compared with empirical equations, including current code equations. As expected, the results show that building periods estimated based on simple equations provided by earthquake design codes in Europe (EC8) and America (UBC97 and ASCE 7-10) are significantly smaller than the periods computed using nonlinear dynamic analysis. Based on the results obtained from the analyzed models, equations for calculating period of vibration are proposed. These proposed equations will allow design engineers to quickly and accurately estimate the fundamental period of moment resisting frames with taking different length to width ratios, vertical element size, floors area and building height into account. The interaction between reduction factor and the reduced period of vibration is studied, and it is found that values of maximum period of vibration can be used as an alternative method to calculate the inelastic base shear value without taking reduction factors in consideration.

Keywords: Fundamental Period of Vibration; Moment Resisting Frames; Stiffness and Mass of Building.

1. Introduction

Determination of fundamental period is essential to earthquake design and assessment. The accurate estimation for this property will improve the estimation of global seismic demands. Since this property is dependent on mass and stiffness, it is affected by many factors such as building material, structural regularity, the number of stories and bays and the section properties including dimensions and extent of cracking. Cracking of RC members decreases its stiffness significantly and so this reduction should be considered in analysis to determine the expected period of vibration.

As this property cannot be analytically computed for a structure before design process, building codes provide empirical formulas that is depend on material, steel or concrete, and height of structure or number of stories. It is also allowed using finite element with assumed mass and stiffness to determine this property during the preliminary design stage and limit the estimated period with an upper bound factor.

*Corresponding author: ahmednader727@gmail.com

DOI http://dx.doi.org/10.28991/cej-2019-03091438

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
The main objectives of this study are: carrying out a parametric study on moment resisting frames (MRF) in terms of height of structure, the ratios of length to width of plan dimensions \((L/W)\), various sections of vertical elements and plan area, comparing the resulted periods with current codes formulae, deriving a new equation to estimate the fundamental period of moment resisting frames by using non-linear regression analysis.

2. Period Height Relationships in Seismic Design Codes

ATC3-06 presented the first simplified equation that relates the period to the height of the building and, for many years, this equation has been used to design reinforced concrete structures with this form [1]:

\[
T = C_t \times H^{0.75}
\]  

(1)

Where, \(T\): fundamental period of vibration; \(C_t\): regression coefficient (0.025 for RC moment resisting frames); \(H\): the height of the building.

SEAOC-88 [2] re-evaluated the value of regression coefficient and it was found that the value of \(C_t=0.03\) was more valid than the value given by ATC3-06 [1]. Generally, the coefficient \(C_t\) was calibrated such that the derived fundamental period would underestimate the period by approximately 10-20% at the first yield of building to obtain conservative estimate for the base shear.

Bertero et al. (1988) studied the time histories of building responses for 18 RC Moment Resisting Frames. Two substantial increases were found in the results. The cause of the first increase was identified to be the onset of structural and non-structural damage. The reason for the second increase in period was non-structural components are no longer contributing significantly to the stiffness of the structure, causing the building to vibrate as a bare structural frame. They concluded that the regression coefficient value 0.04 is more reliable than the coefficient value proposed by SEAOC-88. For a lower bound estimate of the period, Bertero et al. (1988) recommended the use of \(C_t=0.035\) [2, 3].

Bertero et al. (1988) found that the contribution of non-structural elements to stiffness of buildings used in Bertero et al. study is trivial if it is compared with structural frame. It is also noted that non-structural elements do not have a considerable effect beyond the first 5 seconds of earthquake motion. As a result, the equation obtained from Bertero et al. study for design of reinforced concrete moment resisting frames appears to be justified [3, 4].

The use of period-height form with SEAOC-88 [2] recommended 0.03 \((C_t=0.075 \text{ m})\) coefficient has been included in many design codes, for example, UBC-97 [5] and EC-8 [6]. In UBC-97 [5], it is permitted to calculate the fundamental period by using rational analysis but the calculated period shouldn’t exceed 1.3 \(T\) for zone 4 and 1.4\(T\) for zone 1, 2 and 3; where “\(T\)” is the approximated fundamental period determined by Equation 1.

Goel and Chopra (1997) gathered data from 8 Californian earthquakes, starting from the 1971’s San Fernando earthquake and ending with the 1994’s North Bridge earthquake. These data include the measured period in the longitudinal and transverse directions. They showed that codes formulas underestimate fundamental period of moment resisting frames, especially structures more than 60 stories. According to these results, they proposed an alternative period-height formula, Equation 2, with limiting the estimated period by upper bound. ASCE 7-10 prescribed the formula which proposed by Goel and Chopra (1977) in order to estimate the approximated fundamental period of moment resisting frames [7, 8].

\[
T = C_t \times H^x
\]

(2)

In which \(C_t\): regression coefficient \((0.075 \text{ for RC MRFs})\); \(X=0.9 \text{ for RC MRFs}\); \(H\) : height of building.

It is allowed to calculate the fundamental period by using rational analysis but the calculated period should not exceed \(C_a^T\); where “\(T\)” is the approximated fundamental period determined by Equation 2 and “\(C_a\)” is the magnification factor shown in Table 1.

<table>
<thead>
<tr>
<th>Design spectral Response Acceleration (SD1)</th>
<th>Coefficient (C_a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥ 0.4</td>
<td>1.4</td>
</tr>
<tr>
<td>0.3</td>
<td>1.4</td>
</tr>
<tr>
<td>0.2</td>
<td>1.5</td>
</tr>
<tr>
<td>0.15</td>
<td>1.6</td>
</tr>
<tr>
<td>0.1</td>
<td>1.7</td>
</tr>
<tr>
<td>≤ 0.05</td>
<td>1.7</td>
</tr>
</tbody>
</table>
3. Measured and Analytical Fundamental Periods

The instrumentation of buildings provides important information. It increases the knowledge about the actual behaviour of building during earthquakes. The data can be used to verify formulas analytically. A lot of researches have been carried out to determine the vibration properties of buildings from their recorded motions during earthquakes and to evaluate the current analytical modelling with the performance of instrumented buildings.

Hart et al. evaluated the response of reinforced concrete structure with 10 stories located in California during Whittier earthquake. The main lateral resisting system was moment resisting frames. The recorded fundamental period from the strong motion was 1.1 sec. A structural analysis was carried by using ETABS 2015 [9, 10]. The analysis was carried out for three different degrees of cracking. Un-cracked gross section, cracked beams only and fully cracked (beams and columns). Hart et al. (1989) found that the case with fully cracked best correlated with the recorded period during earthquake strong motion with measured period 1.17 sec [9].

Anderson and Bertero (1977) studied the seismic performance of the San Bruno office building. The lateral resisting force system was three ductile moment resisting frames in the longitudinal direction and two ductile moment resisting frames in the transverse direction. The fundamental periods from the Loma Prieta earthquake were 1.05 and 0.85 sec. for the longitudinal and transverse directions, respectively. The identified initial periods due to low amplitude accelerations were 0.71 and 0.57 sec. for the same directions. It was observed that these initial periods correlated well with ambient vibration test results of 0.58 and 0.71 sec; respectively. Undertaking a three dimensional analytical modelling for the building using ETABS 2015 [10] software, Anderson and Bertero (1977) found that the results were in good agreement with the ambient vibration tests when cracking was not considered in modelling, and it was close to the values obtained from strong motion records, Loma Prieta earthquake records, when the rigidity of the columns was decreased to 50%. The estimated fundamental periods by finite element software were 1.05 and 0.85 sec; respectively [10, 11].

Mirtaheri and Salehi (2018) have been attempt to make a comprehensive examination between the fundamental period obtained from ambient vibration tests and those obtained from code provisions and finite element analysis [12]. ISC-2014 [13] and ASCE 7-10 [8] are the codes utilized in their study. Their study was carried out on 17 reinforced concrete buildings. Their study was carried out on 17 reinforced concrete buildings, lateral system of buildings is varied between moment resisting concrete buildings and dual system. Based on the results provided by the investigations general results were achieved: first result, the fundamental period obtained from the ISC-2014 [13] have, on average, a difference of about 18%–38% with those obtained from the tests. This amount of difference may be due to the fact that the empirical relationships which have been suggested in the ISC-2014 [13] to calculate the fundamental period are not sufficiently accurate and need to be reviewed. Second result, the average difference between the fundamental period obtained from the ASCE 7-10 [8] and those obtained from the tests ranges from 16%to 54% for the various lateral resisting systems. Same as before, this can indicate that the empirical relationships of the ASCE 7-10 [8] suggested to calculate the fundamental period are also not accurate enough.

4. Comparison between Numerical Analyses Results and Code Formulas

Al-Balhawi and Zhang (2017) studied MRF systems designed under gravity and wind loadings. The parameters considered include the number of storeys, the number and length of bays, plan configurations, mechanical properties of infill walls, and the presence of openings in the un-cracked and cracked infill wall. They found that periods of vibration of bare frames reasonably agrees with codes formulas by disregarding contributions of infills’ stiffness towards the structural systems. On the other hand, the proposed formulas for RC MRF buildings with un-cracked infills agree well with most cited experimentally based formulas and some numerically based ones [14].

Young and Adeli (2016) investigated fundamental periods of eccentrically braced frame (EBF) structures with varying geometric irregularities. A total of 12 EBFs are designed and analyzed. Based on the results obtained from vibration theory. They found that a 3-variable power model which is able to account for irregularities resulted in a better fit to the Rayleigh data than equations which were dependent on height only. They also compared the resulted periods of vibration with available measured period data and found that their analytical results are closer to the measured periods than current codes equations [15].

Asteris et al. (2016) investigated the results of a large-scale analytical study on the parameters that affect the fundamental period of reinforced concrete structure. They studied parameters were number of storeys, the number of spans, the span length, the infill wall panel stiffness and the percentage of openings within the infill panel. The results were shown to fit better the data than codes equation, having a high correlation factor and a low mean square error and can adequately estimate the fundamental period of masonry-infilled RC buildings [16].

Asteris et al. (2016) used artificial neural networks (ANNs) to predict the fundamental period of infilled reinforced concrete (RC) structures. For the training and the validation of the ANN, a large data set is used based on a detailed investigation of the parameters that affect the fundamental period of RC structures. To comparison of the predicted
values with analytical ones indicates the potential of using ANNs for the prediction of the fundamental period of infilled RC frame structures taking into account the crucial parameters that influence its value [17].

5. The Applied Element Method

The structure is modeled in AEM by creating small elements that are connected together. The two elements are then connected through a group of normal and shear springs that represent the stress and strain around each element Tagel-Din and Meguro (2000) [18]. The main feature of this software is analyzing the model in all stages until it is totally collapsed. It is able also to track behavior of elements during separation, contact and collision. In AEM the connecting springs are representing the tool which is used to gather the elements together. Each spring is actually 3 springs; 1 spring for normal and 2 springs for shear deformation. To model the concrete in compression the Maekawa compression model as shown in Figure 1(a) is adopted. To model concrete in tension, its response is elastic until the formation of cracks occurred. After cracking, it is assumed that concrete subjected to tension loses strength through a softening mechanism and the opened cracks can be represented by a loss of elastic stiffness. To model concrete in shear, shear stress-strain relation remains linear until it reaches the cracking point. After cracking, shear stress descents down as shown in Figure 1(b). The level of drop in shear stresses depends on the interlock of aggregate particles on the two sides of crack and friction at the crack surface. For reinforcement springs, Figure 2 shows the model, presented by Menegotto and Pinto (1973) for cyclic loading of reinforcement steel bars used in AEM [19].

6. Case Study

6.1. Description of the Studied Structure

Studied structure is a typical 10-storey building. The structure consists of four bays in both directions, the typical bay width is five meters in the both directions. The height of each storey is three meters. The columns are assumed to be fixed at their base. Figure 3 shows general layout of the studied building.
6.2. Slab Dimension

The studied cases were designed according to BS EN 1992-1 [20]. So, according to these guidelines the thickness of solid slab with span 5 m is 150 mm. Cube compressive strength concrete ($F_{cu}$) is 30 MPa and yield strength of steel reinforcement ($F_y$) is 360 MPa.

6.3. Gravity and Seismic Loads

Analysis software calculates own weight of members, by multiplying the specific weight by volume of element. The flooring cover, equivalent uniform wall load and uniform live load are assumed to be 2.5, 3.0 and 2.5 KN/m$^2$, respectively. For seismic loads, seismic zone is assumed to be 5B with peak ground acceleration=0.3g. Soil type is assumed to be type C with soil factor=1.5 and the response modification factor equals to 6.0.

6.4. Elements Reinforcement

Figure 4 shows the reinforcement steel bars that are used according to design of the three sections, one at both ends and the third one at the middle, due to gravity loads and seismic loads straining actions:

For the 1st, 2nd and 3rd storeys

[Diagram showing reinforcement for different storeys]
For the 4th, 5th and 6th storeys

For the 7th, 8th, 9th and 10th storeys

Figure 4. Reinforcement details of Beams and Columns (Base case)

7. Description of Case Studies

In this study, effect of building height, length to width ratio (L/W), columns size and area of structure on fundamental period was considered. The study is divided to four groups with four cases for each group, where each group represents the effect of these parameters on the fundamental period. For example, the 1\textsuperscript{st} group includes the effect of building height. Each case has a designation beginning with MRF followed by 1, 2, 3 or 4 representing 1\textsuperscript{st}, 2\textsuperscript{nd}, 3\textsuperscript{rd} and 4\textsuperscript{th} group, respectively; Following is a numerical designation indicating the case number. For example, MRF-1-3 represents that this structure is the third case in the first group. Tables 2 shows the Varied and constant parameters of studied cases.

<table>
<thead>
<tr>
<th>Building Designation</th>
<th>Height (m)</th>
<th>L/W Ratio</th>
<th>Columns size (mm)</th>
<th>Area (m\textsuperscript{2})</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF-1-1</td>
<td>30</td>
<td>1</td>
<td>600x600</td>
<td>400</td>
</tr>
<tr>
<td>MRF-1-2</td>
<td>33</td>
<td>1</td>
<td>600x600</td>
<td>400</td>
</tr>
<tr>
<td>MRF-1-3</td>
<td>36</td>
<td>1</td>
<td>600x600</td>
<td>400</td>
</tr>
<tr>
<td>MRF-1-4</td>
<td>39</td>
<td>1</td>
<td>600x600</td>
<td>400</td>
</tr>
<tr>
<td>MRF-2-2</td>
<td>30</td>
<td>2</td>
<td>600x600</td>
<td>400</td>
</tr>
<tr>
<td>MRF-2-3</td>
<td>30</td>
<td>3</td>
<td>600x600</td>
<td>400</td>
</tr>
<tr>
<td>MRF-2-4</td>
<td>30</td>
<td>4</td>
<td>600x600</td>
<td>400</td>
</tr>
<tr>
<td>MRF-3-2</td>
<td>30</td>
<td>1</td>
<td>800x800</td>
<td>400</td>
</tr>
<tr>
<td>MRF-3-3</td>
<td>30</td>
<td>1</td>
<td>1000x1000</td>
<td>400</td>
</tr>
<tr>
<td>MRF-3-4</td>
<td>30</td>
<td>1</td>
<td>1200x1200</td>
<td>400</td>
</tr>
<tr>
<td>MRF-4-2</td>
<td>30</td>
<td>1</td>
<td>600x600</td>
<td>900</td>
</tr>
<tr>
<td>MRF-4-3</td>
<td>30</td>
<td>1</td>
<td>600x600</td>
<td>1600</td>
</tr>
<tr>
<td>MRF-4-4</td>
<td>30</td>
<td>1</td>
<td>600x600</td>
<td>2500</td>
</tr>
</tbody>
</table>
8. The Stiffness of RC Members in Linear Eigenvalue Analysis

The gross (un-cracked) stiffness is inappropriate in considering cracking of critical elements such as beams which generally cracked under gravity loading only. However, if cracking is not found to have occurred before the design seismic level of excitation (considered unlikely as this level of excitation would with all probability have preceded by a number of lower intensity events), it will occur early on in the response to excitation and thereafter the stiffness will reduce rapidly. Therefore, Codes provided stiffness reduction factors to reflect this reduction in member’s stiffness. The two cases of member’s stiffness, cracked and gross stiffness, have been applied to the case studies and eigenvalue analysis has been done to find the corresponding fundamental period at each case.

9. Time History Records

Each of the studied cases has undergone non-linear dynamic analysis by using Extreme Loading of Structures software (ELS, 2016 [21]). In this study, the design earthquake acceleration equals 0.3g. The ground acceleration is obtained by converting the EC-8 [6] response spectrum into time history function by using SIMQKE program, Gasparini and Vanmarcke (1976) and then this function is imported to (ELS, 2016) [11] to represent the ground motion. The ground acceleration for our case is shown in Figure 5 [22].

10. Determining Period of Vibration from Dynamic Non-Linear Analysis

Codes equations were calibrated according to the period at first yield of buildings which obtained from the available data of these buildings during their motions records. The same criteria were followed here in determining the fundamental period. The first time at which reinforcement springs reach yield stress is determined from Time-Springs stresses chart, shown in Figure 8; and so the period which corresponds to the first time at which reinforcement springs stresses is the yield stress in Time- Period chart, shown in Figure 9; is the period at first yield of building. According to Figure 8, the first time at which reinforcement springs stress reaches yield stress is 4.5 sec, which does actually correspond to the period 1.8 sec in Figure 9. Hence, fundamental period at first yield of this building is 1.8 sec.

Figure 6. Ground Acceleration time history of earthquakes equals to (0.3g)

Figure 7. Time-Reinforcement Springs Stresses Chart in ELS
11. Results

For the first group, height of building increased with keeping mass of building constant, therefore lateral stiffness of building decreased and so the fundamental period increased. The estimated fundamental period is much higher than period calculated by codes.

For second group, in this group length to width ratio increased from 1 to 4, as this ratio increased with constant mass the lateral stiffness of building increased and so fundamental period decreased. The estimated fundamental periods through eigenvalue analysis with cracked stiffness become much closer to period calculated by codes as L/W increased and periods at first yield of building become in the range between estimated periods by codes and their upper bound.

For the third group, size of columns increased with keeping mass of building constant, therefore lateral stiffness of building increased and so the fundamental period decreased. The estimated fundamental periods are higher than period calculated by codes but they are close to estimated fundamental periods through eigenvalue analysis with cracked stiffness.

For the fourth group, area of building increased with keeping lateral stiffness of building constant, therefore total mass of building increased and so fundamental period increased. The estimated fundamental period is much higher than period calculated by codes.

11.1. First Group

The results of this group are shown in Figures 10 and 11. This group clarify the effect of building height on the fundamental period.
11.2. Second Group

The results of this group are shown in Figures 12 and 13. This group clarify the effect of length to width ratio of building on the fundamental period.

Figure 10. Comparison of Period-Height for Period at First Yield of building, Period at the Level of Significant Yield (Max. T), Cracked Eigenvalue Analysis, Gross Stiffness Eigen value Analysis, ASCE 7-10

Figure 11. Comparison of Period-L/W in the two directions for Period at First Yield of building, Period at the Level of Significant Yield (Max. T), Cracked Eigenvalue Analysis, Gross Stiffness Eigen value Analysis, EC-8 and UBC-97
11.3. Third Group

The results of this group are shown in Figures 14 and 15. This group clarify the effect of columns size on the fundamental period.

Figure 12. Comparison of Period- L/W in the two directions for Period at First Yield of building, Period at the Level of Significant Yield (Max. T), Cracked Eigenvalue Analysis, Gross Stiffness Eigen value Analysis, ASCE 7-10

Figure 13. Comparison of Period-Column size for Period at First Yield of building, Period at the Level of Significant Yield (Max. T), Cracked Eigenvalue Analysis, Gross Stiffness Eigen value Analysis, EC-8 and UBC-97
11.4. Fourth Group

The results of this group are shown in Figures 16 and 17. This group clarify the effect of area of building on the fundamental period.
12. Regression Analysis

Firstly, a separate equation is derived for each group as shown in Figures 18 to 20. Then, the derived equations and fundamental period of base case are used to obtain modification factors $C_h, C_r, C_k$ and $C_a$. The following form given in Equation 3 is used in creating equation to estimate fundamental period of vibration based on the results of gross stiffness eigenvalue analysis, cracked stiffness eigenvalue analysis and non-linear dynamic analysis.

$$T = T_0 \times C_h \times C_r \times C_k \times C_a$$  \hspace{1cm} (3)

Where; $T_0$: Fundamental Period of base Case; $C_h$: The result of dividing period-height relationship by $T_0$; $C_r$: The result of dividing period-L/W ratio relationship by $T_0$; $C_k$: The result of dividing period-column length relationship by $T_0$; $C_a$: The result of dividing period-plan area relationship by $T_0$.

![Figure 17. Period-Height Relationship Obtained from Gross Stiffness Eigenvalue Analysis, Cracked Stiffness Eigenvalue Analysis and Non-Linear Dynamic Analysis](image-url)
Figure 18. Period- L/W Ratio Relationship in the two directions Obtained from Gross Stiffness Eigenvalue Analysis, Cracked Stiffness Eigenvalue Analysis and Non-Linear Dynamic Analysis

Figure 19. Period-Column Length Relationships Obtained from Gross Stiffness Eigenvalue Analysis, Cracked Stiffness Eigenvalue Analysis and Non-Linear Dynamic Analysis
The derived equations 4 to 6 are given below to estimate the fundamental period of vibration for moment resisting frames bounded by heights from 30 to 39 meters, L/W ratios from 1 to 4, columns length from 600 to 1200mm ,plan areas from $20 \times 20$ to $50 \times 50$ m$^2$ and Peak ground acceleration=$0.3a_g$. The three equations are derived from gross eigenvalue analysis, cracked eigenvalue analysis and non-linear dynamic analysis, respectively. Second group results in X and Y directions are nearly equal. Then, one direction was taken into account in deriving these equations. The upper limit of the suggested equation for deriving fundamental period is the multiplication of the three Equations 4, 5 and 6 by factor 1.4, as prescribed in seismic codes.

$$T = 0.00040 \times H^{1.24} \times R^{-0.48} \times K^{-0.25} \times A^{0.51}$$  \hspace{1cm} (4) \\
$$T = 0.00077 \times H^{1.22} \times R^{-0.58} \times K^{-0.46} \times A^{0.44}$$ \hspace{1cm} (5) \\
$$T = 0.0012 \times H^{1.37} \times R^{-0.66} \times K^{-0.48} \times A^{0.42}$$ \hspace{1cm} (6)

13. Calculation of Base Shear Forces to Show the Interaction between Reduction Factor and the Reduced Period of Vibration

Design codes prescribed a base shear coefficient to be used in design. This base shear coefficient was given as a proportion of the weight which was to be resisted laterally by the structure. EC-8 [6] prescribed the below formulas to calculate ultimate base shear:

$$F_b = (S_d(T) \times W \times \lambda)/g$$  \hspace{1cm} (7)

$$S_d(T) = a_g \times g \times S \times \frac{2.5}{R} \times \frac{T_C}{T} \geq 0.2 \times a_g \times \gamma_1$$ \hspace{1cm} (8) \\
$$T_C \leq T \leq T_D$$

$$S_d(T) = a_g \times g \times S \times \frac{2.5}{R} \times \left(\frac{T_C}{T} \times T_B\right) \geq 0.2 \times a_g \times \gamma_1$$ \hspace{1cm} (9) \\
$$T_D \leq T \leq 4 \text{ sec}$$

Where; $F_b$: the ultimate base shear-force; $W$: the design weight of structure; $\lambda$: correction factor, for the studied cases its value=1.0; $S_d(T)$: horizontal design spectrum; $a_g$: the designed ground motion; $\gamma_1$: the importance factor, in this study its value=1.0; $R$: the reduction factor which determined according to structural system; $S$, $T_C$ and $T_D$: Coefficients determined according to soil type, for the studied cases $T_C = 0.25$, $T_D = 1.2$ and $S=1.5$.

Base shear forces were calculated by using the above equations according to the period of vibration calculated from Equation 1 and according to the maximum estimated period by using non-linear dynamic analysis. Base shear values were calculated with considering reduction factor equals to one ($R=1$) and without considering the minimum value of base shear.
The purpose of calculating the base shear forces with considering the reduction factor equals to one (R=1) is to determine the ability of the obtained maximum period of vibration to consider structures non-linearity. If the resulted base shear by using values of obtained period of vibration and according to code’s response spectrum and the obtained base shear by ELS [21] are close, this means that the obtained maximum period of vibration can be directly used to estimate inelastic base shear values without taking effect of reduction factor. On the other hand, If the resulted base shear by using values of obtained period of vibration and according to code’s response spectrum are much bigger or smaller than the obtained base shear by ELS [21], this means that effect of reduction factors is independent on the effect of the reduced period of vibration. Figure 21 shows the resulted base shear forces for all cases according to the calculated period by Equation 1 and according to the maximum estimated period by using non-linear dynamic analysis.

Figure 21. Comparison between the resulted base shear forces for MRFs according to the calculated period by EC-8 and according to the maximum estimated period by using non-linear dynamic analysis. Where:

14. Conclusions

Results of the non-linear dynamic analysis are considered to be more realistic as it takes the real effect of cracking into account in estimating stiffness of structures (all members will not be cracked or yielded at the same time). Then, with comparing the results of codes formula with results of nonlinear dynamic analysis the following points are concluded:

- Unless the current form of code formula takes the height of structure into consideration, it gives periods much shorter than the estimated periods. The estimated periods tend to be 3.076 times the code period.

- Area of structure should be considered in code formula for estimating fundamental period of moment resisting frames. The estimated period increased by 116% when the area of structure increased from 400 to 2500 square meters.

- Length to width ratio should be considered in code formula for estimating fundamental period. When the ratio increased from 1 to 4, the estimated period decreased by 62%.
• Sizes of columns should be considered in code formula for estimating the fundamental period of MRFs. The estimated period decreased by 28% when the size of column increased from 600×600 to 1200×1200.

• As the values of base shear considering maximum period of vibration, at level of significant yield, with modification factor equals to one (R=1) is close to the ELS values of base shear, the values of maximum period of vibration can be used as an alternative method to calculate the inelastic base shear value without taking reduction factors in consideration.

15. Conflicts of Interest

The authors declare no conflict of interest.

16. References


An Evaluation of Barriers obstructing the Applicability of Public Private Partnership (PPP) in Infrastructure Development

Sedqi Esmaeel Rezouki a, Jinan Kata’a Hassan a*

aDepartment of Civil Engineering, College of Engineering, University of Baghdad, Baghdad, Iraq.

Received 10 August 2019; Accepted 25 October 2019

Abstract

Shortage in funds after the declining in oil prices since 2014, made Iraq government encourage private sector engagement in financing infrastructure projects through PPP. However, private sector reluctance was notable. Therefore, this research is conducted to assess if Iraq is a supportive environment for PPP projects development. 25 risk factors of PPP projects have been listed and organized within a questionnaire that was conducted with a participation of 98 respondents from public, private institutions and academics. Means comparison was used to rank and identify respondent agreement on assessing the level of importance of these risk factors, also nonparametric tests were used. Findings indicated that all respondents groups have agreed on ranking corruption on the top of barriers that government should deal with to ensure the success of PPP projects. Afterward scarcity of private funds came in the first place followed by insufficient public administration processes and then by the lack of legal framework followed by the delays in acquisition of land while the lack of sovereign guarantee came at the fifth place. The perceptions of survey groups’ respondents concerning the importance of risk factors differ, where both public and academics respondents have serious concerns regarding the private sector capacity to carry out the task and the availability of private funds. On the other hand the private sector concerns the availability of government incentives to support for infrastructure PPP projects. Overall findings indicated that government must work on building a solid enabling environment before the initiation of PPP approach in Iraq.

Keywords: PPP; Enabling Environment; Barriers; Risk Factors; Iraq.

1. Introduction

PPP defined as long-term contractual agreement between a public agency and a partner or consortium of companies from private sector to carry out the design, implementation, financing, operating and management of the infrastructure. In the partnership the responsibility of financing, constructing, asset management and maintenance, and service provision will be on the private partner; in return the private partner will obtain payments from the government and/or from user fees [1]. As PPP agreements depend on private funding it will be an efficient tool for delivering infrastructure and filling the gap between required capital cost and government limited financial resources in addition to cost-effectiveness [2]. A proper risk sharing assumes to transfer risk to the party that best able to control. Accordingly government should not transfer risks that private sector will not be able to control and manage [1]. PPP have been used worldwide in industrialized, industrializing, and developing countries. The purpose of PPPs adoption varies greatly from country to another, in industrializing countries like UK and Germany; PPP adopted in public service provision. Meanwhile industrializing countries, with tremendous needs for basic infrastructure like China and India, PPPs used to sustain rapid economic growth it’s usually seen in power, water or road sectors [3]. In developing countries PPP adopted to overcome

*Corresponding author: jinan_gataa@yahoo.com

http://dx.doi.org/10.28991/cej-2019-03091439

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
the shortage in the government’s financial resources or to fulfill conditions imposed by International Organizations to provide loans [4-7]. In the case of Iraq as a developing country, Government of Iraq (GOI) faced a severe shortage in funding due to the decline in oil prices in 2014, which made GOI suspend near 2700 under-construction infrastructure projects funded under the government capital budget projects [8]. As a solution to prevent destruction of the projects that had reached high percent of completion and to fulfill the shortage in basic services by new project too, the PPP appear to be as appealing solution. Accordingly, government encouraged governmental institutions to engage private sector in financing infrastructure projects that have been suspended due to the financial crisis through public-private sector partnership to be another financing option. However, private entities’ involvement in filling that gap was unpromising. Therefore, this study aims to explore major barriers and risk factors to PPP projects in Iraq using quantitative methods.

The paper consists of the following sections; section 2 discuss advantages & criticisms of the PPP, section 3 discuss the required conditions for successful application of section 4 review PPP experience and major barriers and risk factors impacting PPPs Projects in Middle East and North Africa (MENA) to take advantage of their experience and the similarities in conditions with Iraq as developing countries, section 5 discuss challenges and risks associated with PPP’s contracts in Iraq, section 6 provides details about the applied research approach, section 7 highlighted Results discussions, and finally, section 8 summarizes overall conclusions of this paper.

2. Advantages and Criticisms of the PPP

PPPs as tool for infrastructure projects development and provision coupled with many advantages, which chiefly resulted from PPP major three features: private fund, creativity and proper risk sharing [9]. Figure 1 shows PPP contracts main feature and advantages.

![Figure 1. Main feature and advantages of PPP contracts [10]](image)

On the other hand, critics on using PPPs for infrastructure provision are mainly about:

- Weakness and poor ability of public entities to manage the implementation of PPP projects, high costs of bidding poor ability in contracts design and bids formulation in addition to the need to hiring services’ advisory [11-13].

- The complex nature of these contracts makes it difficult to structure PPP projects and predict its contingencies [14].

- Improper contracts’ design may fail to deal flexibly to make the needed adjustments to boost public authority requirements in the provided services. Moreover, if risks weren’t sufficiently allocated private partners this may lead to costly consequences on public authority [12].

- Added future expenditures resulted fiscal obligations which impose constraint on governments [9, 11].

- Long term contracts may lead to opportunistic behaviour and underinvestment [1].

- As private entities may not be able to borrow money with a low rate of interest as the public agencies leading to higher project capital costs [14]

- Investor’s transparency about returns could be questioned, in addition to liabilities imposed by PPP projects on users and taxpayers [12].

3. Conditions for Successful PPPs

Although PPP considered being a promising approach for infrastructure development and the provision of services in the different countries around the world, PPP could be inadvisable for some government and for some projects. Accordingly, lessons learned from international best practices have identified conditions that should be fulfilled to increases the possibility of a successful PPP implementation [10, 15]. Alternatively, government can achieve value for money through traditional procurement by applying a sound procuring and overseeing procedures and strategies [16].
3.1. Committed Political Leadership

The transition in infrastructure's procurement from traditional public procurement to PPP is an approach adopted by political leadership and be obligated to by the top of governmental hierarchical. If PPP be committed at that level, the mobilization of resources needed for PPP success will be guaranteed. Building mutual trust between the different partners and stakeholders by establishing enabling environment that provide guaranties and safeguard equally for all partners and investors will also lay on the political leadership [17-19].

3.2. Project is Suited to PPP Model

Project that best suits the PPP model should have a number of the following features: 1) adequate number private companies a proper qualification to ensure competitive bidding; 2) innovation possibilities; 3) project is self-financed; 4) provide loop of feedback starting from price setting to a provision of service; 5) possibility of tasks bundling; 6) possibility of proper risk transfer to private partner; 7) the need to obtain private sector specialization that public sector is lacking to; 8) Project outputs' specifications can be well defined and measured; 9) large strategic projects to obtain a possibility of spreading out the capital cost of along the contract term; 10) An adequate preparation time to ensure the contract will be negotiated properly [20-22].

3.3. Institutional Structures and Legal/Regulatory Framework

There are fundamental principles for PPP implementation at the program level which include: 1) The availability of PPP institutional legal framework and Policy; 2) Developing competent PPP units where, the establishment of competent PPP units to deal with PPPs projects represents a substantial element in overcoming institutions' poor qualification and capabilities'; 3) Perception of PPP objectives; 4) Performance and method specifications [23, 24]; 5) Initiating a systematic assessment and revision of current legislation and regulations and the need for development of new ones; and the establishment of a standardized approval's processes and a sound interagency coordination to avoid and eliminate any institutional and/or regulatory barriers to PPP implementation [15], this may implemented on both levels local and national.

3.4. The Proper Risks Sharing

Allocating anticipated risks to the party that best able to manage and control it is the only way to achieve value for money. Furthermore, the lack of transferring enough risk to the private partner, will lead to the lack of motivations to achieve PPP expected objectives [25]. A proper risk sharing represent a major attribute of well-developed public entities managerial and technical abilities [15].

3.5. Law Authority and the Independence of Judiciary System

One of the most attractive factors for investors to undertake PPP projects is the Law authority and the independence of judiciary system in protecting contractual and ownership rights [17]. Another important factor is the clear and timely manner disputes' resolution procedures. Where government must set into place such procedures to ensure a good disputes resolution are available wherever disagreements arise throughout the agreement term [26].

3.6. Public Sector Ability

Government should work on three aspects to ensure a proper implementation of: 1) Consensus building between the different stakeholders including society to ensure acceptance of PPP projects; 2) Sustain an efficient PPP contract management throughout PPP project lifecycle; 3) highly qualified staff legally, financially, and technically apart from political impact to ensure that PPP optimum features risks sharing, value for money, and PPP project monitoring is achieved [20]. Building a well-qualified public entities technically and managerially is a major step in overcoming institutional constraints that may hamper the implantation of PPP projects [15].

4. PPP in Middle East and North Africa (MENA)

In developing countries there are four major themes should be taken into consideration to achieve a successful implementation of PPP projects and gain its favorable benefits; 1) macroeconomic indicators impacts, 2) political situation's influences, 3) organizational structures circumstances, and 4) factors associated to PPP project. It’s important to understand that the elements of these themes are interrelated and all should be dealt with and improved [27] Figure 2, summarize these factors.
MENA countries as other developing countries have unfavorable business environment with high level of potential risks resulted in investors’ reluctance, in addition to the political and security turmoil which have made the situation even worse. This section will focus on six countries in the region which have passed through similar conditions to Iraq and have earlier experience with PPP and passed through conditions similar to the conditions that Iraq has passed through like; Egypt, Tunisia and Morocco, or have a well-developed experience with PPP projects implementation like Turkey, Dubai and Jordan who have recognized the importance of engaging private sectors in the development of infrastructure and started to found the required enabling environment for this participation by establishing the needed legal/regulatory framework to support the success PPP program implementation. The Main findings include:

4.1. PPP Legal and Policy Frameworks in MENA

4.1.1. Turkey

Despite the fragmented legal and institutional structure and improper interagency coordinating and management, Turkey succeeded in developing public entities’ capacities to undertake PPP projects implementation. This can be clearly seen in a good number of PPP projects conducted in the different sectors since the 90s. PPP unit have been established in the Ministry of development (MoD) in 2007. The unit made a significant contribution in the Tenth Development Plan [28]. Table 1 summarizes the number of projects and their value in the six focus countries plus Iraq [29].

<table>
<thead>
<tr>
<th>Country</th>
<th>Jordan</th>
<th>Morocco</th>
<th>Tunisia</th>
<th>Egypt</th>
<th>Turkey</th>
<th>Dubai*</th>
<th>Iraq</th>
</tr>
</thead>
<tbody>
<tr>
<td>Projects reaching financial closure</td>
<td>40</td>
<td>22</td>
<td>9</td>
<td>30</td>
<td>213</td>
<td>8</td>
<td>13</td>
</tr>
<tr>
<td>Sector with largest investment share</td>
<td>water &amp; Sewerage</td>
<td>Ports</td>
<td>water &amp; Sewerage</td>
<td>water &amp; Sewerage</td>
<td>Roads</td>
<td>Social &amp; Health</td>
<td>Ports</td>
</tr>
<tr>
<td>Infrastructure Sectors Reported</td>
<td>ICT &amp; Sewerage</td>
<td>Natural gas</td>
<td>ICT &amp; Sewerage</td>
<td>ICT &amp; Port</td>
<td>All</td>
<td>ICT</td>
<td></td>
</tr>
<tr>
<td>Type of PPI with largest share in investment</td>
<td>Mgt. &amp; lease contracts</td>
<td>Greenfield project</td>
<td>Greenfield project</td>
<td>Mgt. &amp; lease contracts</td>
<td>Mgt. &amp; lease contracts</td>
<td>NA</td>
<td>Greenfield project</td>
</tr>
<tr>
<td>Cancelled Projects</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>% Cancelled Projects of total investment</td>
<td>11%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>9%</td>
<td>0%</td>
<td>0%</td>
</tr>
</tbody>
</table>

* Dubai data between 2013-2018, source: www.infrappworld.com

4.1.2. Jordan

Has a good track and has enacted new PPP law no. 31 of 2014 anticipated to bring greater clarity to the legal framework [29, 30].
4.1.3. Morocco

Has engaged private sector in developing infrastructure by concession. In 2011 the central PPP unit has been established [17]. Moreover, PPP new Law No. 86-12 on PPP contracts enacted on December 24, 2014 followed by Decree no 2-15-45 on May 13th, 2015 providing regulation for PPP law [30].

4.1.4. Tunisia

Has also fragmented legal and institutional structure in implementing PPP projects [17]. In November 27th, 2015 Law No. 49-2015 on PPPs enacted, and PPP unit formulated [30].

4.1.5. Egypt

Has a specific PPP law no. 67 enacted in 2010 [30]. Also a competent and efficient central PPP unit has been established. Despite that, concession contract is still in use according to sectorial statutes [17].

4.1.6. Dubai

Has enacted PPP law no. 22 of 2015, which consider a mature approach to attracting long-term investment to the Emirate [31]. There is no centralized PPP authority according to Dubai’s PPP act. In fact, Dubai’s PPP act gives government entities entering into a PPP arrangement the flexibility to manage the financial, economic and social feasibility of a project, as well as the distribution of risk between the parties [32].

4.2. Major Barriers and Risk Factors Impacting PPPs Projects in MENA

4.2.1. Egypt, Jordan, Morocco and Tunisia

The examination of the enabling environment in these countries throughout 2005 to 2015, it can be concluded that the highest level of potential risks lay within institutional and legal aspects (which include; 1) Lack of Legal framework, 2) Interagency coordination, 3) Poor public decision-making process, 4) Project scoping (incl. contract design & risk allocation), 5) Bidding process, 6) Government Capacity, and 7) The capacity of the private sector) where the highest frequencies of barriers have been indicated. It forms with operational barriers (which include; 1) Land acquisition, 2) Choice of location, 3) Construction risks, and 4) Social & Environmental risks) the larger percent of risks that hindering the PPP progress as presented in Figure 1. The interesting finding, risks related to political considerations (which include; 1) Government stability/ security conditions, 2) Corruption/ lack of transparency, 3) Public understanding & opposition, and 4) Change of law & Breach of contract) indicated low percent than other risks ranged from (10-25) % to Morocco and Tunisia respectively. Meanwhile financial consideration (which include; 1) lack of private funding, 2) Transfer of funds, 3) lack of sovereign guarantee, 4) Inflation rate, 5) Changes in interest rate, 6) Changes in currency exchange rates, and 7) Tax regulation) came after ranged from (10-24) % to Tunisia and Egypt respectively. From Figure 3 it can be noticed that Egypt and Tunisia have higher percent compared to the other countries indicating the impact of political turmoil started in 2011[17].

![Figure 3. Risk factors to PPP in Egypt, Jordan, Morocco, and Tunisia [17]](image)

This could be explained by risks related to political considerations is uncontrolled, changing them will take long time to achieve political stability on the other hand statutory and organizational risks can be under government control and
can be improved quicker than political risks. Accordingly if government urging to succeed in undertaking PPP approach it must adopt a scheduled reforming program that support PPP implementation in the different stages of PPP projects development.

4.2.2. Dubai

Knight (2016) identifies two challenges Dubai needs to deal with in implementing PPP program. Firstly, specify certain sectors to apply PPP, this will enable investors to reduce the cost of bids preparation if they fail in one project they are almost prepared to redirect their efforts to another one within the same sector. Secondly, to overcome oil prices fluctuation that may have high effect on the available liquidity of public authorities. Government should ensure a proper risks sharing and sufficient pricing to attract to foreign funders in filling that gap [33]. Despite Dubai PPP Law is considered an overwhelmingly positive step forward in facilitating PPP projects in the Emirate [32] in more deep examination, the top ten critical success factors to PPP projects in UAE are: 1) Public and private partners commitment, 2) Proper risk sharing, 3) Ability and qualification of public sector entities, 4) Transparency of the processes, 5) Competent private partner, 6) Competitive bidding procedures, 7) Committed political leadership, 8) sound feasibility studies, 9) Proper administration, and 10) Supporting legal/Regulatory Framework [34].

4.2.3. Turkey

Turkey as a developing country in order to succeed in PPP should establish a solid ground to support PPP program initiation and implementation that would include; 1) PPP institutional structure and enabling legal/regulatory framework; 2) Overcoming The struggle of powers within public authorities to ensure effective coordination; 3) Careful study for financial amplifications of suggested PPP to minimize financial consequences to State budgetary; 4) Huge need for infrastructure development may led to aggressive use of PPPs with no consideration for value for money which should be considered; 5) Adequate capacity building program should be provided to governmental entities to implement PPP contracts successfully [28].

In a deeper exploration, studies illustrated that proper institutional structures and legal/regulatory framework, precise selection of project that is suited to PPP model, and sound and comprehensive feasibility studies indicated a higher impact on PPP successful implementation. On the other hand proper consultancy to client, projects uncomplicated structural organization, and social perception and understanding seem to have lower impact [35]. In more generic and brief interpretation there are five areas need to work on to support PPP implementation; 1) PPP project financing, proper management, operational influences, competitive procuring processes, and supportive institutional structure [35].

5. Challenges and Risks of PPP’s Contracts in Iraq

The enabling environment refers to the relevant policies, laws, regulations and institutions which allow and support the development of infrastructure projects, as well as overall government support, capacity and commitment for PPPs in the country [36]. The absence of any of these elements can lead certain themes to emerge hindering PPP progress. Iraq, through the past 15 years the existing PPP projects were procured based on different laws including act No. 22 enacted in 1997 for public firms, the act of firms No. 21 enacted in 1997, Investment act No. 13 of 2006. Where there is no designated PPP law enacted to be the legal base for PPP projects agreement in Iraq which represent a real barrier in PPP progress. In regard to other challenges, Wali (2015) had addressed a generic perception of the most important potential challenges and risks that would face the PPP implementation in Iraq. She suggested government should consider three aspects; firstly, improving government poor experience in PPP. Secondly, fulfilling stakeholder expectations and thirdly, adopting a phased, pragmatic approach [37].

In addition to these aspects the researcher identifies another two serious barriers. The first one is improving business and investment environment. The Transparency International organization ranks Iraq 166th out of 176 countries in its Corruption Perception Index [38]. Iraq also has a poor business climate and stood on place 168 out of 189 in the World Bank’s Doing Business Index 2018 (a drop of 22 place over 2014), with low scores in all the indicators [39]. Table 2 shows the World Bank ranking of doing business for some countries in the region. If Iraq is to build a stronger, more resilient economy, it must improve its business and investment environment, its capacity to attract private investment will be critical in that respect.

<table>
<thead>
<tr>
<th>Economy</th>
<th>UAE</th>
<th>Turkey</th>
<th>Morocco</th>
<th>Tunisia</th>
<th>Jordan</th>
<th>Egypt</th>
<th>Iraq</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rank global (2018)</td>
<td>21</td>
<td>60</td>
<td>69</td>
<td>88</td>
<td>103</td>
<td>128</td>
<td>168</td>
</tr>
<tr>
<td>Scores (2018)</td>
<td>78.73</td>
<td>69.14</td>
<td>67.91</td>
<td>63.58</td>
<td>60.58</td>
<td>56.22</td>
<td>44.87</td>
</tr>
<tr>
<td>Rank global (2014)</td>
<td>25</td>
<td>51</td>
<td>68</td>
<td>56</td>
<td>116</td>
<td>113</td>
<td>146</td>
</tr>
<tr>
<td>Scores (2014)</td>
<td>75.08</td>
<td>68.39</td>
<td>64.43</td>
<td>67.45</td>
<td>58.29</td>
<td>59.17</td>
<td>50.79</td>
</tr>
</tbody>
</table>
The second one is the interagency coordination PPP contracts are so the complex nature of PPP arrangements required efficient and closely working relationships between competent public authorities in on hand and private partners on the other hand.

6. Research Approach

The applied research methodology is presented in Figure 4. Using of the same concept applied in Al-juboori (2015), Li et al. (2005) and Babatunde et al. (2016) [40-42] studies, the study implemented based on a quantitative analysis. A well-structured questionnaire (presented in Appendix I), and based on a quantitative analysis used to identify the standpoints of respondents on the theme of the research presented in this paper. The questionnaire divided into five parts. This paper is targeted to present the two parts of the questionnaire. Part 1 presents respondents’ general data. Part 2 (represent the fourth part of the questionnaire) aims to identify respondent agreement on assessing the level of importance of risk factors related to PPP enabling environment that considered as key barriers may face the development of PPP projects in Iraq which is the target of this paper. Through the literature review of this research, 25 risk factors of PPP projects have been listed under four main groups, political situation concerns, financing, statutory and organizational, and functional risks as shown in Table 3, where their importance will be rated by the survey respondents on Likert six points scale where 1=least important”, 6= very important” with an option of 0= don’t know/ inapplicable.

![Research Methodology Diagram](image-url)
Table 3. Risk factors related to PPP enabling environment in Iraq

<table>
<thead>
<tr>
<th>Risk Category</th>
<th>Code</th>
<th>Risk Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Political</td>
<td>PR1</td>
<td>Government stability/ security conditions</td>
</tr>
<tr>
<td></td>
<td>PR2</td>
<td>Corruption/ lack of transparency</td>
</tr>
<tr>
<td></td>
<td>PR3</td>
<td>Public understanding &amp; opposition</td>
</tr>
<tr>
<td></td>
<td>PR4</td>
<td>Change of law &amp; Breach of contract</td>
</tr>
<tr>
<td>Financial</td>
<td>FR1</td>
<td>Scarcity of private fund in general (local and foreigner)</td>
</tr>
<tr>
<td></td>
<td>FR2</td>
<td>Transfer of funds</td>
</tr>
<tr>
<td></td>
<td>FR3</td>
<td>lack of sovereign guarantee</td>
</tr>
<tr>
<td></td>
<td>FR4</td>
<td>Inflation rate</td>
</tr>
<tr>
<td></td>
<td>FR5</td>
<td>Changes in interest rate</td>
</tr>
<tr>
<td></td>
<td>FR6</td>
<td>Changes in currency exchange rates</td>
</tr>
<tr>
<td></td>
<td>FR7</td>
<td>Tax regulation</td>
</tr>
<tr>
<td>Legal &amp; institutional</td>
<td>LR1</td>
<td>Lack of Legal and regulatory framework</td>
</tr>
<tr>
<td></td>
<td>LR2</td>
<td>Interagency coordination</td>
</tr>
<tr>
<td></td>
<td>LR3</td>
<td>Insufficient public administration processes</td>
</tr>
<tr>
<td></td>
<td>LR4</td>
<td>Lack of government support and incentives</td>
</tr>
<tr>
<td></td>
<td>LR5</td>
<td>Project scoping (incl. contract design &amp; risk allocation)</td>
</tr>
<tr>
<td></td>
<td>LR6</td>
<td>Bidding process</td>
</tr>
<tr>
<td></td>
<td>LR7</td>
<td>Lack of commitment of public sector</td>
</tr>
<tr>
<td></td>
<td>LR8</td>
<td>Government Capacity</td>
</tr>
<tr>
<td></td>
<td>LR9</td>
<td>The capacity of the private sector</td>
</tr>
<tr>
<td></td>
<td>LR10</td>
<td>Lack of commitment of private sector</td>
</tr>
<tr>
<td>Operational</td>
<td>OR1</td>
<td>Delays and problems associated with acquisition of land</td>
</tr>
<tr>
<td></td>
<td>OR2</td>
<td>Choice of location</td>
</tr>
<tr>
<td></td>
<td>OR3</td>
<td>Construction risks</td>
</tr>
<tr>
<td></td>
<td>OR4</td>
<td>Social &amp; Environmental risks</td>
</tr>
</tbody>
</table>

6.1. Questionnaire Distribution and Data Collection

150 questionnaires survey are distributed using separate questionnaire hard copies as well as an online questionnaire using google drive forms and distributed to public sector organizations work in the infrastructure projects development, private sector companies, private banks and academics. 116 questionnaires were returned which form (77%) of the total distribution. On the other hand 98 questionnaires are answered where 18 of the returned questionnaires deemed ineligible due to improper survey’s respondent, blanked answers, ineligible, and multiple answers. The percent of questionnaire returned and valid (84%) is considered adequate for the purpose of analysis and reporting based [43].

6.2. Data Analysis

The 98 returned valid questionnaires were analyzed using Statistical Package of Social Science (SPSS) version 25. Respondents’ general information was analyzed using descriptive analysis. As the Normality test have been checked and data set is not following a normal distribution, the Kendall’s Coefficient of Concordance is used to test the internal agreement between the respondents within the same group; and Kruskal Wallis test is conducted to test the agreement among respondents of the survey groups. The data is analysed in three-stages,

1) Identifying risk importance depending on the values of their means with ordinal organization from the highest to the lowest.
2) Kendall's $\mathcal{W}$ concordance coefficient test will be used to examine inner agreements within the same group.
3) Determining the differences between any two groups’ participants’ ratings on any individual risk factor, through applying Kruskal Wallis and Mann–Whitney U tests.
7. Results and Discussions

7.1. Respondents General Information and Rate of Response

Table 4 shows the general information of the respondents. Public sector respondents form 56% of survey respondents, while private sector form 27% and academics form 17%. In regard the general work experience, respondents who have general work experience not less than 21 years form 50% of the whole respondent followed by those with general work experience of 16-20 years by 29.6% then those with general work experience of 11-15 years by 14.3% and those who have general work experience of 6-10 years by 5.3%. While those with general work experience of 5 years or less were only 1%. Findings show that those who have at least 21 years of general working experience form majority of public, private sectors and academics respondents by 43.6%, 57.7%, and 58.8% respectively. Indicating that the target respondents have an adequate experience that to provide a balanced view and reliability for the research survey. In relation to the years of experience in PPP projects implementation, finding shows that 53% of respondents from the public sector are lacking the experience in PPP projects, forming the highest percent within the three groups of respondents, followed by academics respondents with 41.2%. Meanwhile the percentage of respondents from the private sector who have no previous experience is 30.8%. Furthermore, the lowest percentage is 10.9% of public sector’s respondents, who have over 6 years’ of experience. As the implementation of PPP projects is not conducted widely and limited to a few number projects in Iraq [29], it is expected to find that more than one-half (53%) of respondents from the public sector have no previous experience in PPP.

<table>
<thead>
<tr>
<th>Respondents profile</th>
<th>Percentage of respondents in the survey sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Sector of work</td>
<td>Overall</td>
</tr>
<tr>
<td></td>
<td>100%</td>
</tr>
<tr>
<td>2 Years of Experience in work</td>
<td>Overall</td>
</tr>
<tr>
<td></td>
<td>5 Years or less</td>
</tr>
<tr>
<td></td>
<td>6-10 Years</td>
</tr>
<tr>
<td></td>
<td>11-15 Years</td>
</tr>
<tr>
<td></td>
<td>16-20 Years</td>
</tr>
<tr>
<td></td>
<td>Over 21 year</td>
</tr>
<tr>
<td>3 Years of experience in PPP projects implementation</td>
<td>Overall</td>
</tr>
<tr>
<td>Non</td>
<td>44.9%</td>
</tr>
<tr>
<td>1-2 years or less</td>
<td>13.3%</td>
</tr>
<tr>
<td>3-5 years</td>
<td>21.4%</td>
</tr>
<tr>
<td>Over 6 years</td>
<td>20.4%</td>
</tr>
</tbody>
</table>

7.2. Result on to Risk Factors Importance Ranking

Mean (M) and standard deviation (SD) values obtained using SPSS illustrated in Table 5 in addition to risk factors ranking as viewed by the sample respondents in the three groups. It can be concluded from Table 5 that the scores are close to the mean since SD of all rated factors is relatively small. It can also be noted that the M scores values of all factors ranged between 4.13-5.76 which means all the tested factors considered important based the ratings of all participants since it vary from important to highly important to ensure a successful implementation to PPP adoption.

It can be seen from Table 5, that the means values for the factors as rated by the respondents from public sector are ranged from 4.13 to 5.60, which indicate a relatively small variance in the responses by (1.47). The means values of private sector and academics respondents are from 4.27 to 5.73 and 4.41 to 5.76 respectively. Means values differences are also small by (1.46) and (1.62) respectively. The small differences in means shown in the survey groups indicate that survey respondents have rated these factors much more consistently.
From the perspective of all respondents, findings from Table 5 indicate that the main risks challenges to PPP in Iraq as it have been agreed on by all respondents is “Corruption” the following top five most important barriers and risk factors (in descending order) that may prohibit the implementation of PPP projects in Iraq are:

- Scarcity of private fund in general (local and foreigner);
- Insufficient public administration processes;
- Lack of Legal framework;
- Delays and problems associated with acquisition of land.

“Corruption/lack of transparency in public administrative” is ranked on the top of risks factors and barriers that government should deal with to ensure the success of PPP projects. It is not surprising that overall respondents have ranked corruption as on the top of risks factors hampering PPP in Iraq’s business environment, as Iraq stood on the order 167th out of 176 countries in its Corruption Perception Index [38]. Corruption represents the main barrier not for PPP implementation only but in the country development process and stability.

The second one is “private funding’s Scarcity”. Financial scarcity may lead to termination of the project and losing the invested funds. Act of Investment no. 13 enacted in 2006 and its amendments, provide incentives to risks related to political and market demand risks to attract private investors as well as government liabilities or repayment obligations, in addition to provision of exemption related to certain taxes, which justify the relative low rating of “Taxing system” FR7 and “Exchange rates fluctuation” FR6, this doesn’t mean these factors are satisfying investors expectation as the taxing system and legislations is too old and need to be modernized by considering Tax as an economic tool employed properly in encouraging foreign and local investors and the private sector in general in compliance with provision of

---

**Table 5. Relative importance of risk factors for PPP projects in Iraq rated by the survey respondents**

<table>
<thead>
<tr>
<th>Code</th>
<th>Public Mean</th>
<th>Public SD</th>
<th>Public Rank</th>
<th>Private Mean</th>
<th>Private SD</th>
<th>Private Rank</th>
<th>Academic Mean</th>
<th>Academic SD</th>
<th>Academic Rank</th>
<th>Overall Mean</th>
<th>Overall SD</th>
<th>Overall Rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>PR1</td>
<td>5.20</td>
<td>0.70</td>
<td>9</td>
<td>5.35</td>
<td>0.797</td>
<td>6</td>
<td>5.65</td>
<td>0.606</td>
<td>2</td>
<td>5.32</td>
<td>0.726</td>
<td>8</td>
</tr>
<tr>
<td>PR2</td>
<td>5.60</td>
<td>0.596</td>
<td>1</td>
<td>5.73</td>
<td>0.667</td>
<td>1</td>
<td>5.76</td>
<td>0.562</td>
<td>1</td>
<td>5.66</td>
<td>0.608</td>
<td>1</td>
</tr>
<tr>
<td>PR3</td>
<td>4.82</td>
<td>0.796</td>
<td>21</td>
<td>5.12</td>
<td>0.816</td>
<td>13</td>
<td>5.47</td>
<td>0.800</td>
<td>7</td>
<td>5.01</td>
<td>0.831</td>
<td>16</td>
</tr>
<tr>
<td>PR4</td>
<td>5.05</td>
<td>0.989</td>
<td>12</td>
<td>4.77</td>
<td>0.992</td>
<td>19</td>
<td>5.12</td>
<td>1.054</td>
<td>18</td>
<td>4.99</td>
<td>1.000</td>
<td>17</td>
</tr>
<tr>
<td>FR1</td>
<td>5.60</td>
<td>0.596</td>
<td>2</td>
<td>5.31</td>
<td>0.788</td>
<td>7</td>
<td>5.35</td>
<td>0.862</td>
<td>12</td>
<td>5.48</td>
<td>0.707</td>
<td>2</td>
</tr>
<tr>
<td>FR2</td>
<td>4.27</td>
<td>1.079</td>
<td>24</td>
<td>4.42</td>
<td>0.987</td>
<td>23</td>
<td>4.82</td>
<td>1.237</td>
<td>23</td>
<td>4.41</td>
<td>1.092</td>
<td>24</td>
</tr>
<tr>
<td>FR3</td>
<td>5.45</td>
<td>0.571</td>
<td>5</td>
<td>5.42</td>
<td>0.758</td>
<td>5</td>
<td>5.35</td>
<td>0.931</td>
<td>13</td>
<td>5.43</td>
<td>0.689</td>
<td>6</td>
</tr>
<tr>
<td>FR4</td>
<td>5.05</td>
<td>0.756</td>
<td>13</td>
<td>5.04</td>
<td>0.774</td>
<td>15</td>
<td>5.18</td>
<td>0.728</td>
<td>17</td>
<td>5.07</td>
<td>0.750</td>
<td>13</td>
</tr>
<tr>
<td>FR5</td>
<td>4.64</td>
<td>1.060</td>
<td>23</td>
<td>4.69</td>
<td>1.087</td>
<td>21</td>
<td>4.53</td>
<td>1.007</td>
<td>24</td>
<td>4.63</td>
<td>1.049</td>
<td>23</td>
</tr>
<tr>
<td>FR6</td>
<td>4.13</td>
<td>1.001</td>
<td>25</td>
<td>4.27</td>
<td>0.874</td>
<td>25</td>
<td>4.41</td>
<td>1.121</td>
<td>25</td>
<td>4.21</td>
<td>0.987</td>
<td>25</td>
</tr>
<tr>
<td>FR7</td>
<td>5.00</td>
<td>0.770</td>
<td>14</td>
<td>4.92</td>
<td>0.628</td>
<td>18</td>
<td>5.06</td>
<td>0.966</td>
<td>20</td>
<td>4.99</td>
<td>0.767</td>
<td>18</td>
</tr>
<tr>
<td>LR1</td>
<td>5.55</td>
<td>0.689</td>
<td>3</td>
<td>5.23</td>
<td>0.765</td>
<td>9</td>
<td>5.41</td>
<td>1.064</td>
<td>11</td>
<td>5.44</td>
<td>0.787</td>
<td>4</td>
</tr>
<tr>
<td>LR2</td>
<td>5.13</td>
<td>0.668</td>
<td>10</td>
<td>5.19</td>
<td>0.567</td>
<td>10</td>
<td>5.41</td>
<td>0.618</td>
<td>9</td>
<td>5.19</td>
<td>0.637</td>
<td>11</td>
</tr>
<tr>
<td>LR3</td>
<td>5.36</td>
<td>0.729</td>
<td>6</td>
<td>5.69</td>
<td>0.549</td>
<td>2</td>
<td>5.47</td>
<td>0.717</td>
<td>6</td>
<td>5.47</td>
<td>0.692</td>
<td>3</td>
</tr>
<tr>
<td>LR4</td>
<td>5.07</td>
<td>0.920</td>
<td>11</td>
<td>5.50</td>
<td>0.762</td>
<td>4</td>
<td>5.47</td>
<td>0.874</td>
<td>8</td>
<td>5.26</td>
<td>0.889</td>
<td>9</td>
</tr>
<tr>
<td>LR5</td>
<td>4.89</td>
<td>0.956</td>
<td>18</td>
<td>5.19</td>
<td>1.021</td>
<td>11</td>
<td>5.29</td>
<td>0.772</td>
<td>14</td>
<td>5.04</td>
<td>0.952</td>
<td>14</td>
</tr>
<tr>
<td>LR6</td>
<td>4.89</td>
<td>0.994</td>
<td>17</td>
<td>5.15</td>
<td>1.084</td>
<td>12</td>
<td>5.24</td>
<td>0.752</td>
<td>16</td>
<td>5.02</td>
<td>0.984</td>
<td>15</td>
</tr>
<tr>
<td>LR7</td>
<td>4.98</td>
<td>0.991</td>
<td>15</td>
<td>5.27</td>
<td>0.667</td>
<td>8</td>
<td>5.41</td>
<td>0.870</td>
<td>10</td>
<td>5.13</td>
<td>0.904</td>
<td>12</td>
</tr>
<tr>
<td>LR8</td>
<td>5.27</td>
<td>0.849</td>
<td>8</td>
<td>4.96</td>
<td>1.148</td>
<td>17</td>
<td>5.59</td>
<td>0.795</td>
<td>3</td>
<td>5.24</td>
<td>0.942</td>
<td>10</td>
</tr>
<tr>
<td>LR9</td>
<td>5.53</td>
<td>0.690</td>
<td>4</td>
<td>5.00</td>
<td>0.894</td>
<td>16</td>
<td>5.53</td>
<td>0.800</td>
<td>5</td>
<td>5.39</td>
<td>0.795</td>
<td>7</td>
</tr>
<tr>
<td>LR10</td>
<td>4.93</td>
<td>0.766</td>
<td>16</td>
<td>4.35</td>
<td>0.936</td>
<td>24</td>
<td>5.29</td>
<td>1.047</td>
<td>15</td>
<td>4.84</td>
<td>0.916</td>
<td>20</td>
</tr>
<tr>
<td>OR1</td>
<td>5.33</td>
<td>0.668</td>
<td>7</td>
<td>5.54</td>
<td>0.582</td>
<td>3</td>
<td>5.59</td>
<td>0.618</td>
<td>4</td>
<td>5.43</td>
<td>0.642</td>
<td>5</td>
</tr>
<tr>
<td>OR2</td>
<td>4.85</td>
<td>0.705</td>
<td>19</td>
<td>5.08</td>
<td>0.891</td>
<td>14</td>
<td>5.12</td>
<td>0.697</td>
<td>19</td>
<td>4.96</td>
<td>0.759</td>
<td>19</td>
</tr>
<tr>
<td>OR3</td>
<td>4.69</td>
<td>0.979</td>
<td>22</td>
<td>4.73</td>
<td>1.041</td>
<td>20</td>
<td>4.94</td>
<td>1.144</td>
<td>22</td>
<td>4.74</td>
<td>1.019</td>
<td>22</td>
</tr>
<tr>
<td>OR4</td>
<td>4.84</td>
<td>0.714</td>
<td>20</td>
<td>4.65</td>
<td>0.846</td>
<td>22</td>
<td>5.00</td>
<td>0.707</td>
<td>21</td>
<td>4.82</td>
<td>0.751</td>
<td>21</td>
</tr>
</tbody>
</table>
article 25 and 26 of the Iraqi constitution regarding the reformation of Iraqi economic and encouragement of investments in the different sectors.

The third risk factor of PPP projects, as respondents generally recognized, is "insufficient public administration processes". In general, inefficient management will lead to increased costs, in addition to undefined and lack of transparency processes will raise the level of uncertainty for investors and eventually increase the cost or loss the interest in the project.

For example in Iraq if you want to start a project it need to pass by 11 steps in average and a total time period of 77 days, costing about 117% of income per capita. Meanwhile UAE the process will pass by 6 step with total time to complete is only 13 day, costs 6% of income per [39].

Fourth barrier is the “Lack of Legal and regulatory framework” as there is no designated PPP law enacted to be the legal base for this type of contracting methods. The government tried to provide a legal cover for that purpose within the provisions of article (16), (15) and (14) of the State Budget Law of 2015, 2016, and 2017 respectively also the Council of Ministers’ decision No. 96 of 2016 and the PPP guidelines and project selection criteria issued by the MoP. Unfortunately provisions of PPP article have been removed from the State Budget Law of 2018 and 2019 due to political oppositions leaving the PPP without legal cover.

The fifth one is “Delays and problems associated with acquisition of land” which represent a significant barrier to PPP projects in developing countries, in particular problems related to squatters in addition to unsolved land ownership disputes for PPP project purposes [27] where any delay even if in a small part project area can lead to a delay in the whole timetable and scheduling as well as project feasibility [44].

### 7.2.2. Risk Factors' Importance Depending on Participants' Point of View in Each Group of the Survey

From Table 6, generally the three groups have agreed that Corruption stood in the top of the risk factors in Iraq. Afterward public sector respondents have ranked “Scarcity of private fund” at the second order. Meanwhile private sector and academics respondents ranked this barrier lower at seventh and twelfth order respectively. From perspective of public sector this can be explain due to private sector reluctance from submitting to PPP infrastructure projects despite the actions that government have taken to encourage private sector entities to finance the implementation of under-construction infrastructure projects by issuing the Council of Ministers’ decision No. 340 of 2015, which authorize the governmental institutions to negotiate with the contractors for the completion of the projects by them or through a second financier to complete the remaining works of the projects that have high percent of completion with an interest rate not exceeding 10%. On the other hand the lower ranking of this factor by respondents from both private sector and academics can be explained by their believe that private fund is available but the problem is the lack of confidence in the Government's commitment to fulfil its obligations under PPP contracts as oil represents the main resource of the government budget which always is affected by the fluctuation in the oil markets, the government ability to fulfill these obligations will be questioned since government has no future vision of Iraqi cash status, to set timetables for payments.

Furthermore, the respondents from both private sector and academics have ranked “Insufficient public administration processes” and “Government stability/ security conditions” respectively at the second place. From the private sector perspective the current institutional procedures and process is not fulfilling their expectations as long as the routine and the complex procedures are still exist which is economically consuming for time and money. Meanwhile the academics ranking of government stability/ security conditions explained by their realization for the role stable political will in supporting PPP approach and attracting foreign investors as the security conditions have been improved.

<table>
<thead>
<tr>
<th>Rank</th>
<th>Public Sector</th>
<th>Private Sector</th>
<th>Academics</th>
<th>Overall respondents</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Corruption/ lack of transparency in public administrative</td>
<td>Corruption/ lack of transparency in public administrative</td>
<td>Corruption/ lack of transparency in public administrative</td>
<td>Corruption/ lack of transparency in public administrative</td>
</tr>
<tr>
<td>2</td>
<td>Scarcity of private fund in general (local and foreigner)</td>
<td>Insufficient public administration processes</td>
<td>Government stability/ security conditions</td>
<td>Scarcity of private fund in general (local and foreigner)</td>
</tr>
<tr>
<td>3</td>
<td>Lack of Legal and regulatory framework</td>
<td>Delays and problems associated with acquisition of land</td>
<td>Government Capacity</td>
<td>Insufficient public administration processes</td>
</tr>
<tr>
<td>4</td>
<td>The capacity of the private sector</td>
<td>Lack of government support and incentives</td>
<td>Delays and problems associated with acquisition of land</td>
<td>Lack of Legal framework</td>
</tr>
<tr>
<td>5</td>
<td>lack of sovereign guarantee</td>
<td>lack of sovereign guarantee</td>
<td>The capacity of the private sector</td>
<td>Delays and problems associated with acquisition of land</td>
</tr>
<tr>
<td>6</td>
<td>Insufficient public administration processes</td>
<td>Government stability/ security conditions</td>
<td>Insufficient public administration processes</td>
<td>lack of sovereign guarantee</td>
</tr>
</tbody>
</table>
The third barrier as ranked by public sector’s respondents is “Lack of Legal and regulatory framework” where, a well-developed PPP regulation that consistent with the Iraqi’s laws and regulations is needed to eliminates any crosses in related laws and can facilitate and clarify any ambiguities regarding the types of PPP contracts, contract forms and the requirements of each type. The need of such regulation is highly increased when there is a limited experience with PPP as in the case of Iraq especially within public entities that will be responsible about the whole process management. As they perceive that the legal cover to PPP projects in the provided by government within State Budget Law of 2015, 2016, and 2017 through the provisions of articles 16, 15 and 14 respectively. And the issued guidelines and regulation for PPP projects implementation by the Ministry of Planning (MoP) was not enough to support and clarify any ambiguities related to this new type of contracts in regard to infrastructure projects. this lack became more outstanding when Parliament for political reasons disapproved the inclusion of PPP article in the State Budget Law of 2018 and 2019 which have obstruct the use of PPP.

On the other hand the lower ranking of this factor by respondents from both private sector and academics at the ninth and eleventh place respectively can be explained based on the perception that after 2003, government worked on developing an enabling business environment by changing the legal and regulatory frame to bring in foreign investors and encourage local investors as well. By 2005, the Ministry of Industry adopted a promising program under firms act No. 22, by selecting 36 state owned factories and firms with a share of 51% of a project total ownership and a share of the production too, encouraging by that private investors to engage in management and operation partnership. In 2006, the government had legislated investment act no. 13 which has been amended twice in 2010 and 2015 to give more clarification to major principles of investments, and providing better assurances. Under the 2nd amendment investors are allowable to own lands. Accordingly, undertaking project under Build Operate Transfer (BOT), Build Own Operate Transfer (BOOT) arrangements became available options. Moreover, as leasing for long period (limited to 50 years) is allowable under this act, PPP arrangements such as Building Leasing Operating Transferring (BLOT) and Building Transferring Operating (BTO) became applicable too (investment law 2nd amendment No.50 of 2015).

Meanwhile, the third place as ranked by the respondents from both private sector and academics is “Delays and problems associated with acquisition of land” and “Government Capacity” respectively. Reflecting private sectors concerns regarding the secure access of land and assets which represent a prominent concern and obstructing investors’ participation construction industry Iraq. On the other hand the academics ranking for government capacity as public organizations have a big role throughout the different stages PPP projects’ lifecycle accordingly PPP units must be established and supported with the adequate capacity building programs to handle the management of PPP development and implementation to overcome any institutional weakness before the initiation of PPP program.

The fourth barrier as ranked by the respondents from public sector is “The capacity of the private sector” and at the fifth place by the academics respondents indicating realistic fears of both public sector and academic respondents from the consequences if the private partner fails in fulfilling his obligations. Meanwhile respondents of private sector placed it lower at sixteenth place, ranking instead of at the fourth place the “Lack of government support and incentives” as they believe that PPP infrastructure projects are not feasible if implemented as an investment opportunity unless being supported financially by government. Meanwhile academic respondent have ranked delays and problems associated with acquisition of land on the fourth place agreeing with private sector on its important impact on hampering PPP projects implementation in particular and any construction project in general.

The fifth barrier as ranked by respondents from both the public and private sector is “Lack of sovereign guarantee” reflecting their understanding for the importance of providing sovereign guarantee, such as bond with long maturity term that suit PPP project will provide significant governmental backed risk. And private entities and investors will be encouraged to benefit of these backing guarantee. Meanwhile academics respondents have ranked this barrier lower at the thirteenth place reflecting their concerns regarding the exposer of the government to payment risk on sovereign and/or sub-sovereign borrowers/guarantors.

In general it can be concluded, that public respondents ranking is reflecting their deep understanding for what should work and focus on to build enabling environment for the development of PPP projects. Participants from private entities reflect their concerns about factors under the control of government that have high impact on sustainability of cash flow for PPP projects. Meanwhile academic tend to have more balanced vision that can see things form the different sides resulted in impartial ranking.

Furthermore, as we have mentioned before since the research used Likert scale from 1 to 6, accordingly a value that’s above 3.5 indicate that the barrier is important. Findings show that all barriers are above a mean of 3.5 they are all important to deal with in order to improve the applicability of PPP in Iraq.

### 7.2.3. Testing Inner Agreement within Each Group

Kendall’s $W$ concordance coefficient will be used for this purpose. The suppositions are; zero supposition, where $H_0: W=0$, alternative supposition, $H_a: W≠0$. As tested number of attributes is greater than seven, the value of Chi-square
would be considered instead \( W \) value. It can be seen from Table 7 that the Chi-square critical value for each one of the three groups is 33.20 at degrees of freedom (df) = 24. As the calculated Chi-square values for the three groups of the survey, obtained by the outputs of SPSS are all larger than Chi-square critical value as shown in Table 7, indicating an agreement between the respondents within the same group on these rankings’ of risks.

### Table 7. Kendall’s \( W \) test results on risk ranking of PPP projects

<table>
<thead>
<tr>
<th>Item</th>
<th>Public sector</th>
<th>Private sector</th>
<th>Academics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of survey respondents</td>
<td>55</td>
<td>26</td>
<td>17</td>
</tr>
<tr>
<td>Kendall's ( W )</td>
<td>0.179</td>
<td>0.200</td>
<td>0.154</td>
</tr>
<tr>
<td>df</td>
<td>24</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>Asymp. Sig.</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Chi-Square</td>
<td>236.922</td>
<td>124.796</td>
<td>62.863</td>
</tr>
<tr>
<td>Critical value Chi-square</td>
<td>33.20</td>
<td>33.20</td>
<td>33.20</td>
</tr>
</tbody>
</table>

7.2.4. Testing Inner Agreement within the Sample groups

For this purpose Kruskal Wallis test will be used. Where the supposition are:

Zero supposition, \( H_0 \) : \( \mu_G\text{1} = \mu_G\text{2} = \mu_G\text{3} \)

Alternative supposition, \( H_a \) : \( \mu_G\text{1} \neq \mu_G\text{2} \neq \mu_G\text{3} \)

Where \( \mu_G\text{1}, \mu_G\text{2}, \mu_G\text{3} \) are the median values for public sector (Group 1) private sector (Group 2), and the academics (Group 3) respectively, Table 8 presents obtained outputs from SPSS.

It can be concluded from Table 8 that there are differences in participants’ rating within groups of the sample on PR3 (Public understanding & opposition), LR9 (The capacity of the private sector) and LR10 (Lack of commitment of private sector) which came from the different their understandings and work interest of the participants. This means that the rest 22 factors are rated almost at the same way by all sample groups, with a level of confidence equal to 95%

### Table 8. Kruskal Wallis test for risk factors among survey groups

<table>
<thead>
<tr>
<th>Risk factors</th>
<th>Chi-Square (H value)</th>
<th>Asymp. Sig. (p-value)</th>
<th>Significant</th>
<th>Risk factors</th>
<th>Chi-Square (H value)</th>
<th>Asymp. Sig. (p-value)</th>
<th>Significant</th>
</tr>
</thead>
<tbody>
<tr>
<td>PR1</td>
<td>5.662</td>
<td>0.059</td>
<td>No</td>
<td>LR3</td>
<td>4.200</td>
<td>0.122</td>
<td>No</td>
</tr>
<tr>
<td>PR2</td>
<td>2.736</td>
<td>0.255</td>
<td>No</td>
<td>LR4</td>
<td>5.932</td>
<td>0.052</td>
<td>No</td>
</tr>
<tr>
<td>PR3</td>
<td>9.545</td>
<td>0.008</td>
<td>Yes</td>
<td>LR5</td>
<td>3.493</td>
<td>0.174</td>
<td>No</td>
</tr>
<tr>
<td>PR4</td>
<td>2.732</td>
<td>0.255</td>
<td>No</td>
<td>LR6</td>
<td>2.453</td>
<td>0.293</td>
<td>No</td>
</tr>
<tr>
<td>FR1</td>
<td>2.936</td>
<td>0.230</td>
<td>No</td>
<td>LR7</td>
<td>3.369</td>
<td>0.185</td>
<td>No</td>
</tr>
<tr>
<td>FR2</td>
<td>3.338</td>
<td>0.188</td>
<td>No</td>
<td>LR8</td>
<td>4.540</td>
<td>0.103</td>
<td>No</td>
</tr>
<tr>
<td>FR3</td>
<td>0.045</td>
<td>0.978</td>
<td>No</td>
<td>LR9</td>
<td>8.230</td>
<td>0.016</td>
<td>Yes</td>
</tr>
<tr>
<td>FR4</td>
<td>0.397</td>
<td>0.820</td>
<td>No</td>
<td>LR10</td>
<td>13.877</td>
<td>0.001</td>
<td>Yes</td>
</tr>
<tr>
<td>FR5</td>
<td>0.256</td>
<td>0.880</td>
<td>No</td>
<td>OR1</td>
<td>3.191</td>
<td>0.203</td>
<td>No</td>
</tr>
<tr>
<td>FR6</td>
<td>1.528</td>
<td>0.466</td>
<td>No</td>
<td>OR2</td>
<td>2.342</td>
<td>0.310</td>
<td>No</td>
</tr>
<tr>
<td>FR7</td>
<td>0.930</td>
<td>0.628</td>
<td>No</td>
<td>OR3</td>
<td>1.305</td>
<td>0.521</td>
<td>No</td>
</tr>
<tr>
<td>LR1</td>
<td>3.903</td>
<td>0.142</td>
<td>No</td>
<td>OR4</td>
<td>3.470</td>
<td>0.176</td>
<td>No</td>
</tr>
<tr>
<td>LR2</td>
<td>2.582</td>
<td>0.275</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As Kruskal Wallis test refer to the existence of the difference without specifying in which group, Mann–Whitney–Wilcoxon will be used to identify that using below suppositions:

Zero supposition, \( H_0 \) : \( \mu \text{G1} = \mu \text{G2} \)

Alternative supposition, \( H_a \) : \( \mu \text{G1} \neq \mu \text{G2} \)

Where \( \mu \text{G1} \) and \( \mu \text{G2} \) are the Mean values of score for G1/ public and G2/ Private and G3/ Academics, zero supposition assumes equal mean between paired groups. If \( p \)-value is not larger or equal to 0.05, zero supposition not accepted, as the two groups under the study focus indicates a significant difference.
Where the level of confidence ($\alpha$) will be equal to 0.05 and will be modified and divided over 3 to get rid of I error. Thus, the 3 test will be significant at level equal to 0.0167.

Based on the results of Mann-Whitney test presented in Table 9. It can be seen that, G1 and G2 differs significantly on LR9 with (0.007) and LR10 with (0.008) on the other hand G1 and G3 differs significantly on PR3 with 0.003 in addition to G2 and G3 which also differs significantly on LR10 with 0.001. With end of this test no further tests is required both tests (Mann–Whitney & Kruskal Wallis) conformed each other.

<table>
<thead>
<tr>
<th>Risk Factor</th>
<th>Group 1 and 2</th>
<th>Group 1 and 3</th>
<th>Group 2 and 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mann-Whitney U</td>
<td>Wilcoxon W</td>
<td>Z</td>
</tr>
<tr>
<td>PR1</td>
<td>623.50</td>
<td>2163.50</td>
<td>-1.002</td>
</tr>
<tr>
<td>PR2</td>
<td>609.00</td>
<td>2149.00</td>
<td>-1.344</td>
</tr>
<tr>
<td>PR3</td>
<td>572.00</td>
<td>2112.00</td>
<td>-1.538</td>
</tr>
<tr>
<td>PR4</td>
<td>579.00</td>
<td>930.00</td>
<td>-1.479</td>
</tr>
<tr>
<td>PR5</td>
<td>576.50</td>
<td>927.50</td>
<td>-1.616</td>
</tr>
<tr>
<td>FR1</td>
<td>655.50</td>
<td>2195.50</td>
<td>-0.628</td>
</tr>
<tr>
<td>FR2</td>
<td>668.50</td>
<td>2235.50</td>
<td>-0.210</td>
</tr>
<tr>
<td>FR3</td>
<td>660.00</td>
<td>2200.00</td>
<td>-0.584</td>
</tr>
<tr>
<td>FR4</td>
<td>645.00</td>
<td>996.00</td>
<td>-0.816</td>
</tr>
<tr>
<td>FR5</td>
<td>547.00</td>
<td>898.00</td>
<td>-1.925</td>
</tr>
<tr>
<td>FR6</td>
<td>684.00</td>
<td>2224.00</td>
<td>-0.355</td>
</tr>
<tr>
<td>FR7</td>
<td>536.00</td>
<td>2076.00</td>
<td>-2.057</td>
</tr>
<tr>
<td>FR8</td>
<td>526.00</td>
<td>2066.00</td>
<td>-2.064</td>
</tr>
<tr>
<td>FR9</td>
<td>577.50</td>
<td>2117.50</td>
<td>-1.466</td>
</tr>
<tr>
<td>FR10</td>
<td>591.50</td>
<td>2131.50</td>
<td>-1.316</td>
</tr>
<tr>
<td>OR1</td>
<td>467.50</td>
<td>818.50</td>
<td>-2.663</td>
</tr>
<tr>
<td>OR2</td>
<td>597.00</td>
<td>2137.00</td>
<td>-1.329</td>
</tr>
<tr>
<td>OR3</td>
<td>570.00</td>
<td>2150.00</td>
<td>-1.134</td>
</tr>
<tr>
<td>OR4</td>
<td>602.50</td>
<td>953.50</td>
<td>-1.225</td>
</tr>
</tbody>
</table>

8. Conclusions

1) The main risk challenge to PPP in Iraq as it has been agreed on by all respondents is “Corruption” that government should stop or minimize to the lowest level to improve the government performance in general and in PPP in particular. The following top five risks factors are:
   - Lack of private funding in general
   - Insufficient public administration processes
   - Lack of Legal and regulatory framework
   - Delays and problems associated with acquisition of land
   - Lack of sovereign guarantee

2) The perceptions of survey groups respondents concerning the importance of risk factors differ based on their point of view, where both public and academics respondents have serious concerns regarding the private sector
capacity to carry out the task. On the other hand the private sector concerns the availability of government incentives to support for this type of infrastructure.

3) Improving the capacity of the public sector and other stakeholders by intense and specialized capacity building programs is a priority. Contracting with specialized agencies to support the public organizations with the required expertise and provide in-job training to run the process is a favored option in this initial stage.

4) Legislating PPP law as soon as possible to cover this type of contract, in addition to the establishment PPP units, the development of a comprehensive regulatory framework combined with specific and clear selection criteria of the best private partner.

5) Reforming and standardizing approvals procedures wherever possible, delegate the authorities wherever necessary, and improve the interagency coordination is extremely important to save time and support the application of PPP approach.

6) Risk factors and barriers identified in the enabling environment of Iraq necessitate government commitment and intervention to provide the different types of support including issuing long term bonds to encourage the participation of the private banks and private sector and find a solution to the delay and the problems of land acquisition to attract experienced, well reputation and eligible international firms to ensure good performance and empower the local private sector firms through side by side work enrollment. And in the top of all that minimizing corruption and increasing transparency.

9. Acknowledgments

This study is conducted with a great help, cooperation and support of the Ministry of Construction and Housing and Public Municipalities, Ministry of Planning, and the National Investment Commission.

10. Conflicts of Interest

The authors declare no conflict of interest.

11. References


2658


Appendix I

A Survey on Public-Private Partnership as an Alternative to Financing Suspended Infrastructure Projects in Iraq

To the Respondent

Dear Madam / Sir,

This questionnaire survey is prepared to be a part of a master thesis entitled “Public-Private Partnership as an Alternative to Financing Suspended Infrastructure Projects in Iraq”, submitted to the college of engineering of university of Baghdad in partial fulfilment of the requirements for the degree of Master of Science in civil engineering / project management. The purpose of this study is to assess whether the use of PPP as governmental approach in filling the financing gap is a viable solution for Financing Suspended Infrastructure Projects due to the financial crisis of 2014 and the inability of government to provide all of the required funds for infrastructure development in Iraq.

PPP can be defined as “an agreement between a government and a private firm under which the private firm delivers an asset, a service, or both, in return for payments contingent to some extent on the long-term quality or other characteristics of outputs delivered”. PPPs in facilities development involve private companies in the design, financing, construction, ownership and operation of a public sector utility for long term contract (20-30) year. PPPs are known worldwide with various other alternative names such as Private Participations in Infrastructure (PPI), Private-Sector Participation (PSP), P3, Privately Financed Projects (PFP), and Private Finance Initiatives (PFI).

The survey contains 13 questions, and we estimate it will take an average of 25 minutes to be completed. Your completion of this survey is voluntary and questions are individual, subjective assessments. Your participation in this survey renders me a highly appreciated assistance. Please be sure that your personal data are going to be top confidential.

I welcome your comments or questions relating to this survey, you can contact me at the bellow mentioned addresses.

Notice: It is important to note that there are definitely no “right” or “wrong” answers; the only “correct” answers are what you honestly think and feel.

Thank you in advance for your help, we do appreciate your time.

Jinan k. Hassan
MSc. Student
College of engineering
University of Baghdad
E-mail: jinan_gataa@yahoo.com
### Section 1: Respondent's general information

<table>
<thead>
<tr>
<th>Age:</th>
<th>○ 20-30 years</th>
<th>○ 31-40 years</th>
<th>○ 41-60 years</th>
<th>○ Above 60 years</th>
</tr>
</thead>
</table>

| Name of company/organization: |  |
|-----------------------------|  |

| Your position in the company/organization: |  |
|--------------------------------------------|  |

| Email Address: |  |
|----------------|  |

| Phone Number: |  |
|---------------|  |

**Please select your main role below:**

- How many years of work experience do you have?
  - ○ 5 years or below
  - ○ 6 – 10 years
  - ○ 11 – 15 years
  - ○ 16 – 20 years
  - ○ 21 years or above

| Which sector do you have experience with? |  |
|-----------------------------------------|  |
  - ○ Public sector (State)
  - ○ Private sector

### Section 2: General Experience with PPP

**1. How many years of PPP experience do you have?**
- ○ None
- ○ 1-2 years or below
- ○ 3 –5 years
- ○ 6 years or above

**2. Was there or is there any PPP project undertaken by your company/organization?**
- ○ Yes
- ○ No
- ○ No sure

**3. If yes, is what the type of the project that undertaken by your company/organization (you may tick more than one box)?**
- ○ Transportation
- ○ Roads & Bridges
- ○ Health & Environment
- ○ Water and Sewer
- ○ School and Education
- ○ Other (please specify)
- ○ Housing
- ○ Power and Energy

**4. What is the contract type of the PPP project that undertaken by your company/organization (you may tick more than one box)?**
- ○ Design and build (DB)†
- ○ Lease contract (LC)‡
- ○ Service contract**
- ○ Build-Operate-Transfer (BOT)††
- ○ Build-Own-Operate (BOO)‡‡
- ○ Other (please specify)
- ○ Concession model‡‡
- ○ Management contract‡

**5. Which of the following projects do you think are best suited for PPP projects in Iraq (you may tick more than one box)?**
- ○ All Projects
- ○ High risk projects§
- ○ Governmental infrastructure***
- ○ Social infrastructure‡
- ○ Projects with subsidy¶
- ○ Economical infrastructure††
- ○ Other (please specify)

---

† It is one of the most common contracts in the current government projects. Through the design / construction contract, the owner contracts with one company. According to this contract, the design and construction works are carried out by this company. This company can complete the entire work, or work with sub-contractors through a specific agreement. Design standards must meet the owner's requirements.

‡ This type of contract is called Greenfield, where a private partner in a public-private joint venture constructs and operates a new facility for a specified period of the project contract. The facility may be returned to the Government at the end of the concession period, ownership, operation, capital expenditure and operating expenses shall be the responsibility of the private sector or jointly. These contracts also include contracts: BLO (Build -Lease- Owen) Build-Lease-Own (BOT) Build-Operate- Transfer (BOO) Build-Operate-Operate. Such as oil refinery projects, airports and others.

‡‡ The private partner shall manage, operate, capitalize and run the operating expenses of state-owned enterprises within a certain period with significant investment risks.

‡‡ This type of contract is specifically designed for a management contract or long-term lease contract for the processing of a particular public service through negotiation and contracting with a specialized private company. The private partner manages and operates. The general partner handles capital and property expenditures, either private or public. Such as privatized projects according to the Investment Law.

** Social infrastructure: These structures are linked to the provision of physical assets and services for human development, in sectors such as education, public housing, health care and security (e.g. prisons and rehabilitation centres)

¶ The government may decide to provide direct support to the project, for example through direct financial support (in cash or in kind, to bear construction costs, purchase land, provide assets, compensate for bid costs, or support major maintenance), or waive fees. (Such as exemptions from tax exemptions or waiver of tax liability), financing for the project in the form of loans (including mezzanine debt) or equity investment (in the form of financing) Feasibility gap), and finance the shadow definitions of roads And raise the tariffs paid by some or all consumers (in particular, the least able to pay) as in water and electricity projects to reduce the risk of demand borne by the project company. These mechanisms are particularly useful when the project itself does not achieve debt sustainability or financial feasibility or exposure to certain risks so that investors or private lenders are not well placed for management. In developing countries where private funding is most needed, these constraints may require more government support than is required in more developed countries.

§ High-risk projects are highly visible projects that have a comprehensive impact within and outside the organization and pose significant threats to the project team's ability to implement coupled with high probability of achieving profits or returns.

*** Government infrastructure: includes the provision of facilities to provide services to citizens and administrative centres.

††† Economic infrastructure refers to the provision of physical assets and services related to economic growth, such as sanitation, energy, transit, transport, ports, railways, bridges and highways.
### Section 3: Criteria and Performance of PPP in Iraq

This part aims to study and evaluate the level of awareness and knowledge and current condition of Iraq public sector. Please rate the following statements based on a Likert scale from 1 – 6, where (1= completely disagree; 6 = completely agree; 0= inapplicable).

#### 6. Do you think that PPP is a viable solution for an accelerated public infrastructure projects in Iraq?
- ○ Yes
- ○ No
- ○ No sure

#### 7. Do you think it is more suitable to:

<table>
<thead>
<tr>
<th></th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suspended under-construction infrastructure projects implemented under government capital budget?</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Development of new infrastructure projects?</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 8. Has your organization or company contracted under the PPP Implementation Guide and the Project Regulations issued by the Ministry of Planning in July 2016?
- ○ Yes
- ○ No
- ○ No sure

#### 9. If the answer to question 21 is ”No” (not contracted under this guide), to what extent do you agree with the following points as being impediments for not applying PPP to finance and complete suspended under-construction projects?

<table>
<thead>
<tr>
<th></th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lack of a solid mechanism for PPP projects feasibility studies preparation in government organizations.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>There is no competent office (PPP units) have been established in governmental organizations in both levels of national and local government, provided with the needed capacity building on PPP to run and manage the process.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lack of clear selection criteria for private partner.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lack of a clear and specific mechanism for negotiation, risk identification and better allocation of responsibilities between the two sectors in the different types of PPP.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lack of political commitment to mobilize the needed resources and the establishment of supporting legal and regulatory framework to the success of PPP.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>The issued PPP guidelines and regulations are complicated not easy to be understood by public sector organizations resulted in weak ability for implementation.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lengthy approvals procedures as it related to higher authorities, which requires more time.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PPP law hasn’t been legislated yet to cover this type of contract.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>The majority of suspended projects are related to obstacles and pending problems (including contractors' unpaid payments) that must be resolved first before setting PPP transition agreement.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Private sector reluctance to submit for PPP infrastructure projects as they are not feasible if implemented as an investment opportunity unless being supported financially by government.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Some projects required additional costs as their previous feasibility study is not prepared in accordance to PPP from technical and economic aspects.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Government has no future vision of Iraqi Cash status, to set timetables for payments.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Scarcity of local private funding</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 10. Do you consider your organization is prepared and has the knowledge and the capacity to get involved in a PPP project in regard to the following points?

<table>
<thead>
<tr>
<th></th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Identifying PPP project</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Identifying risks &amp; allocating risks and responsibilities</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PPP project appraisal (project feasibility, commercial viability, whether PPP will provide value for money, whether PPP is fiscally responsible)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Designing PPP contracts (performance requirements, payments mechanism, finance options, adjustment mechanism, dispute resolution mechanism)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Managing PPP Transaction (tendering, evaluation, negotiation, contracting)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Managing PPP contract (establishing contract management structure, monitoring and managing PPP delivery and risk, deal with change)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 11. Considering the following points, do you think that the current legal framework suitable for PPP projects?

<table>
<thead>
<tr>
<th></th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Institutional Structures and Legal/Regulatory Framework.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

* You can see this guide on this link: https://www.dropbox.com/s/shzqzzezwhk/?uii3/AAD3zdN2zx1KZUsckfdeOya?dl=0

---
The Independence of judiciary from the influence of government and politics. □ □ □ □ □ □ □
Law authority and the independence of judiciary system. □ □ □ □ □ □ □
Availability of a proper regulations specifying PPP types, contracts’ templates and required conditions. □ □ □ □ □ □ □
The effectiveness of government initiatives for institutional reforms to support PPP development; e.g. that is related to tariffs and tolling to be paid for the services provided. □ □ □ □ □ □ □

Section 4: Barriers and obstacles that may encounter the implementation PPP.

This part aims to study and evaluate the importance of major perceived Barriers associated with Iraq’s PPP projects. You are asked to circle the number indicating the importance index (1 = Least important; 6 = Most important; 0= not applicable)

12. Considering the following points, how do rate the importance of major barriers and obstacles associated with Iraq’s PPP Infrastructure Projects and the ability to solve or mitigate?

<table>
<thead>
<tr>
<th>Risk factors in PPP projects</th>
<th>Importance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Political</td>
<td></td>
</tr>
<tr>
<td>Government stability/ security conditions</td>
<td>0 1 2 3 4 5 6</td>
</tr>
<tr>
<td>Corruption/ lack of transparency</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Public understanding &amp; opposition</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Change of law &amp; Breach of contract</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Financial</td>
<td></td>
</tr>
<tr>
<td>Scarcity of private fund in general (local and foreigner)</td>
<td>0 1 2 3 4 5 6</td>
</tr>
<tr>
<td>Transfer of funds</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Lack of sovereign guarantee</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Inflation rate</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Changes in interest rate</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Changes in currency exchange rates</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Tax regulation</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Legal &amp; institutional</td>
<td></td>
</tr>
<tr>
<td>Lack of Legal and regulatory framework</td>
<td>0 1 2 3 4 5 6</td>
</tr>
<tr>
<td>Interagency coordination</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Insufficient public administration processes</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Lack of government support and incentives</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Project scoping (incl. contract design &amp; risk allocation)</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Bidding process</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Lack of commitment of public sector</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Government Capacity</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>The capacity of the private sector</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Lack of commitment of private sector</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Operational</td>
<td></td>
</tr>
<tr>
<td>Delays and problems associated with acquisition of land</td>
<td>0 1 2 3 4 5 6</td>
</tr>
<tr>
<td>Choice of location</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Construction risks</td>
<td>□ □ □ □ □ □ □</td>
</tr>
<tr>
<td>Social &amp; Environmental risks</td>
<td>□ □ □ □ □ □ □</td>
</tr>
</tbody>
</table>
**Section 5: Future Prospects**

This part is the end of the survey it aims to obtain participants suggestion and vision to improve PPP implementation in Iraq.

13. Please provide any suggestions and comments that you see is important to improve applying PPP in implementing and financing suspended under-construction infrastructure projects or new projects (please specify)

« End of the questionnaire »

« Thank you for your valuable cooperation»
A New Approximate Method for Earthquake Behaviour of Worship Buildings

Pınar Usta a*, Özgür Bozdağ b

a Isparta Applied Science University, Civil Engineering Department, 32200, Isparta, Turkey
b Dokuz Eylül University, Civil Engineering Department, 35400, İzmir, Turkey

Received 27 March 2019; Accepted 12 September 2019

Abstract

Turkey is in seismically active region, so many earthquakes occur in this country in the last decades. Ancient worship buildings are vulnerable to seismic activity, as many historical buildings. So, it is important to understand that building’s behavior under seismic actions. In this paper, fifteen masonry worship building has been selected which are located and built-in different region in Antalya. The main reason for the paper is to assess the seismic vulnerability of worship building by using a new approximate method. The method which is proposed in this paper aims at a simple and fast procedure based on a simplified geometric approach for immediate screening of masonry buildings at risk.

Keywords: Worship Buildings; Seismic Risk Assessment; Simplified Approximate Method.

1. Introduction

Historical structures have a very important role to carry cultural inherit of the country and they are one of the most valuable pieces of cultural accumulation [1]. There are many historical buildings, religious monuments and ruins of our ancestors [2]. Many historical buildings are quite vulnerable because they were built with low resistance materials. However, these buildings have insufficient connections between the various construction parts; masonry walls, floors, etc. [3]. These problems of historical masonry buildings lead to an overturning collapse of the perimeter walls under seismic horizontal acceleration. For this reason, seismic vulnerability assessments are very important and essential to care for historic masonry structures [4].

Turkey is located on one of the most active several tectonic plates that name is he Alpine–Himalayan earthquake belt. This plate is still active, and many earthquakes occur each month. The city center of Antalya, lying in the second seismic zone of Turkey. When the province is considered in general, the western part of Antalya located in the 1st and 2nd-degree seismic zone, the eastern part of located within the 3rd and 4th-degree seismic zone [5-6]. Antalya is the fifth biggest city in Turkey according to the population. The population of Antalya is approximately 1.2 million. Besides, Antalya is the first rank according to the population growth rate in Turkey. So it is very important to know seismicity of Antalya [7, 8]. Turkey Earthquake Regions Map and Seismic zones map of Antalya is shown in Figure 1.

*Corresponding author: pinarusta@sdu.edu.tr

http://dx.doi.org/10.28991/cej-2019-03091440

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
The approach followed by Lourenço et al. (2013) [10] suggested here is simple and fast being based on a simplified geometric approach for immediate screening of a large number of historical buildings at seismic risk. The aim of the approximate method is to evaluate historical buildings at possible seismic risk, using structural characterization and screening of a large number of historical worship buildings under risk, immediately. The approximate method is applied for historical worship buildings in Antalya, providing lower bound formulas for 10 different simplified geometrical indexes. In this paper, six worship buildings from Antalya have been selected and analyzed considering ten indexes of the approximate method.

2. Approximate Method of Worship Buildings

The approximate method, which is based on the study of Lourenço and Roque (2006), proposed in aims at much more fast and simple procedure for immediate screening of the worship buildings [11]. The analysis and preservation of historical worship buildings are complex phenomena which include many studies. Because of lack of various information such as geometry data and formation about the inner core of the structural elements, existing damage, regulations, and codes about masonry buildings. Moreover, materials that are used in the construction of masonry buildings, that exhibit large variability due to workmanship and that use of natural materials. Therefore, more reliable and better results achieved are not related to more complex and accurate methods [4, 12, 13].

This approximate method is based on a simplified analysis of the structural characteristics of the worship buildings. Each building is inspected individually by its geometry and data is collected which can use for analysis. The usage of the approximate method usually requires that the worship building is regular and symmetric, that the floors act as rigid diaphragms, and that the dominant collapse mode is an in-plane shear failure of the walls. Generally, these last two conditions are not verified by ancient masonry structures [4]. The proposed method consists of ten parameters, and every index has a limit value. Besides ten parameters, defined a total parameter value in this proposed method. Parameters of the approximate method are given below.

• **Parameter 1 (γ₁)** - **In-plan area ratio**

This parameter relates to (being associated with) the area of the earthquake-resistant walls in each main direction (transversal y and longitudinal x, with respect to the central axis of the worship building) and the total in-plan area of the building. Parameter 1 is non-dimensional and the simplest one among the other parameters. The formula of the first parameter, (γ₁), is as follows:

\[ γ₁, i = \left( \frac{A_{wi}}{S} \right) / 0.1 \]

Where \( A_{wi} \) is the in-plan area of earthquake-resistant walls in direction “i” and S is the total in-plan area of the building.

• **Parameter 2 (γ₂)** - **Area to weight ratio**

This parameter is defined as the ratio between the in-plan area of earthquake-resistant walls in each main direction (again, Y and X) and the total weight of the construction. The formula of the second parameter, (γ₂), is as follows:
\[ \gamma_2 \cdot i = \left( \frac{A_{wi}}{G} \right) / 3.25 \]  
(2)

Where \( A_{wi} \) is the in-plan area of earthquake-resistant walls in direction “i” and \( G \) is the quasi-permanent vertical action.

- **Parameter 3 (\( \gamma_3 \)) - Base shear ratio**

  The total design base shear for rigid structures in a given direction shall be determined from the formula is as follows:

  \[ F_E = V_{sd, base} = 0.5ZIw \]  
  (3)

  Where \( Z \) is the seismic zone for the building site, \( I \) is structure importance coefficient, \( w \) is the total seismic dead load.

  The total base shear for seismic loading \( (V_{Sd, base} = F_E) \) can be obtained from an analysis with horizontal static loading equivalent to the seismic action \( (F_E = \beta G) \), where \( \beta \) is an equivalent seismic static coefficient related to the peak ground acceleration. The shear strength of the structure \( (V_{Rd, base} = F_{Rd}) \) can be obtained from the contribution of all earthquake-resistant walls \( F_{Rd,i} = \Sigma A_{wi} f_{vk} \cdot f_{vk} = f_{vk0} + 0.4\sigma_d \). Here, \( f_{vk0} \) is the cohesion, which can be assumed equal to a low value or zero in the absence of more information, \( \sigma_d \) is the design value of the normal stress, and 0.4 describes the tangent of a constant friction angle \( \phi \), equal to 22°.

  The index, \( \gamma_3 \), is as follows:

  \[ \gamma_3 \cdot i = \left( \frac{F_{Rd,i}}{F_E} \right) \]  
  (5)

  If a zero cohesion is assumed \( (f_{vk0} = 0) \), \( \gamma_{3,i} \) is as follows:

  \[ \gamma_{3,i} = \left( \frac{V_{Rd,i}}{V_{Sd}} \right) = \left( \frac{A_{wi}}{A_w} \times \tan \theta \right) / \beta \]  
  (6)

  For a non-zero cohesion, which is most relevant for low height buildings, \( \gamma_{3,i} \) is as follows:

  \[ \gamma_{3,i} = \left( \frac{V_{Rd,i}}{V_{Sd}} \right) = \frac{A_{wi}}{A_w} \times \left( \tan \theta + \frac{f_{vk0}}{(\gamma Xh)} \right) / \beta \]  
  (7)

  Where \( A_{wi} \) is the in-plan area of earthquake-resistant walls in direction “i,” \( A_w \) is the total in-plan area of earthquake-resistant walls, \( h \) is the (average) height of the building, \( \gamma \) is the volumetric masonry weight, \( \phi \) is the friction angle of masonry walls, and \( \beta \) is an equivalent static seismic coefficient.

  Here, it is assumed that the normal stress in the walls is only due to their self-weight, i.e. \( \sigma_d = \gamma \times h \), which is on the safe side and is a very reasonable approximation for historical masonry buildings, usually made of very thick walls. Here, it was assumed that all the masonry materials were similar, the volumetric weight of masonry was 20 kN/m³.

- **Parameter 4 (\( \gamma_4 \)) - Slenderness ratio of columns**

  Parameter 4 is related to the geometric ratio of columns and main wall. \( \gamma_{4,i} \) is as follows:

  \[ \gamma_4 = \left( \frac{h_{col}}{(\gamma Xh)} \right) / 70 \]  
  (8)

  Where \( h_{col} \) is the free height of the columns, \( I \) and \( A \) are the inertia and the cross-section area of the columns.

- **Parameter 5 (\( \gamma_5 \)) - Thickness to height ratio of columns**

  Parameter 5 is related to Thickness to height ratio of columns. \( \gamma_{5,i} \) presented:

  \[ \gamma_5 = \left( \frac{d_{col}}{h_{col}} \right) / 0.05 \quad \text{or} \quad \gamma_5 = \left( \frac{t_{col}}{h_{col}} \right) / 0.05 \]  
  (9)

  Where \( d_{col} \) and \( t_{col} \) are the (equivalent) diameter and thickness of the columns, respectively.

- **Parameter 6 (\( \gamma_6 \)) - Thickness to height ratio of perimeter walls**

  Parameter 6 is related to Thickness to height ratio of perimeter walls. \( \gamma_{6,i} \) is as follows:

  \[ \gamma_6 = \left( \frac{t_{wall}}{h_{wall}} \right) / 0.02 \]  
  (10)

  Where \( t_{wall} \) and \( h_{wall} \) are the thickness and the (average) height of the perimeter walls, respectively.

- **Parameter 7 (\( \gamma_7 \)) - dome area to structure area**

  Parameter 7 is related to the dome area to structure area. \( \gamma_{7,i} \) is as follows:

  \[ \gamma_7 = \left( \frac{K_a}{S} \right) / 0.03 \]  
  (11)

  Where \( K_a \) is an area of dome, \( S \) is an area of worship building.
Parameter 8 ($\gamma_8$) - Dome diameter to dome height

Parameter 8 is related to dome diameter to dome height. $\gamma_8$,i is as follows:

$$\gamma_8 = \frac{K_c}{h_k}/1.3$$  \hspace{1cm} (12)

Where $K_c$ is the diameter of the dome, $h_k$ is height of dome.

Parameter 9 ($\gamma_9$) - Cavity wall area to full wall area

Parameter 9 is related to Cavity wall area to full wall area. $\gamma_9$,i is as follows:

$$\gamma_9 = \left(\frac{A_{wi}}{A_{widolu}}\right)/1.65$$  \hspace{1cm} (13)

Where $A_{wi}$ is the in-plan area of earthquake-resistant walls in direction, $A_{widolu}$ is the in-plan area of earthquake-resistant cavity walls in direction.

Parameter 10 ($\gamma_{10}$) - The ratio of external load base shear force capacity building (dynamic analysis)

Parameter 10 is related to the ratio of external load base shear force capacity building (dynamic analysis). $\gamma_{10}$,i is as follows:

$$\gamma_{10} = \left(\frac{V_{rd}}{V_{sd}}\right)/1.2$$  \hspace{1cm} (14)

Where $V_{sd}$ is the total base shear for seismic loading, $V_{rd}$ is the shear strength of the structure.

Total Parameter

The total parameter is used in determining whether historical building risky or not. Total parameter is as follows:

$$\sum \text{parameter} = \gamma_1 + \gamma_2 + \gamma_3 + \gamma_7 + \gamma_8 + \gamma_{10}$$  \hspace{1cm} (15)

According to the total parameter formulate, which are given above, was used to calculate the risk levels of the worship building. The risk levels are classified as “no risk” and “risk”.

3. Worship Buildings in Antalya, Turkey

In this paper, six historical worship buildings have been selected which are located in Antalya. The worship buildings were explained below.

3.1. Suleymaniye Mosque

The Suleymaniye Mosque is located in Alanya, Antalya. The mosque is also called as Alâeddin, Alâüddin, Kale, Orta Hisar, and Sultan Suleyman Mosque. The mosque had been restored by the General Directorate of Foundations in 1960, 1964, 1973 and 1989. Suleymaniye Mosque consists of octagonal platform and has one main dome. There is a minaret on the northwest corner of the mosque and a five-eyed last community place on the north [14]. Photo and plan of Suleymaniye Mosque shown in Figure 2.

Figure 2. Photo and plan of Suleymaniye Mosque
3.2. Bali Bey Mosque

Bali Bey Mosque located in Muratpasa, Antalya. The Mosque is constructed by Bali Bey according to some resources, but the construction date of the mosque is unknown. The mosque had been restored by the General Directorate of Foundations in 1905, 1963 and 1980. Bali Bey mosque has a rectangular plan, which is close to square, covered with a single dome. And there is the last community room that extending along the northern frontier of mosque. Photo and plan of Bali Bey Mosque shown in Figure 3.

![Figure 3. Photo and plan of Bali Bey Mosque](image)

3.3. Murat Pasha Mosque

Murat Pasha Mosque located in Muratpasa, Antalya. Although it was built in the Ottoman period in 1570, the mosque also has Seljuk calligraphy art traces. According to the inscription the mosque constructed by Murat Pasha. The mosque has a rectangular plan, which is close to square, covered with a single dome. Murat Pasha mosque located in a spacious courtyard and courtyard dimensions is 95×98 m. Photo and plan of Murat Pasha Mosque shown in Figure 4.

![Figure 4. Photo and plan of Murat Pasha Mosque](image)

3.4. Tekeli Mehmet Pasha Mosque

The Tekeli Mehmet Paşa Mosque is a mosque in the city of Antalya, Turkey. Mosque takes its name from Lala Mehmed Pasa. The mosque is constructed in the 18th century in the Kalekapisi district, the mosque is one of the most important Ottoman mosques in the city. Today, the architecture of the mosque, called "Tekeli Mehmet Pasha", "Mehmet Pasha", "Tekeli Pasha", and the construction date are unknown. Photo and plan of Tekeli Mehmet Pasha Mosque shown in Figure 5.
3.5. Omer Pasha Mosque

Omer Pasha Mosque was constructed by Ketendji Omer Pasha in 1602. The mosque is located in Elmali, Antalya Province, Turkey. It reflects the classical Ottoman architecture. The mosque is the biggest Ottoman mosque in the Antalya area. The mosque has a square plan and covered with a central dome. A five-eyed congregation, a fountain, and a madrasah is located the north of the mosque. In the northwest corner, the minaret, which is built adjacent to the harim wall, rises. Photo and plan of Omer Pasha Mosque shown in Figure 6.

3.6. Nasreddin Mosque

Nasreddin mosque is 22 km from the Kas accident of Antalya province. It is located in the village of Kasaba. According to the mosque inscription; the mosque was constructed in 1776 by Yusuf aga. The mosque has a square plan and is covered with a single dome. The mosque has a three-eyed congregation in the north and a minaret in the north-western part. Photo and plan of Nasreddin Mosque shown in Figure 6.
3.7. Musellim Mosque

Musellim mosque, located in Kısla, Antalya, is also known as Teklioglu mosque. According to the mosque inscription the mosque was built in 1796 by Mehmet Aga. Musellim mosque has a square plan and is covered with a single dome. The mosque was restored by the General Directorate of Foundations in 1952, 1955, 1985, 1989 and 1991. Photo and plan of Musellim Mosque shown in Figure 7.

![Figure 7. Photo and plan of Musellim Mosque](image)

3.8. Agalar Onu Mosque

Agalar Onu mosque is located in Aksu, Antalya. According to the mosque inscription; the mosque was constructed in 1776 by Yusuf aga. The mosque has a square plan and is covered with a single dome. The mosque, which is functioning today, was restored by the General Directorate of Foundations in 2011. Photo and plan of Agalar Onu Mosque shown in Figure 8.

![Figure 8. Photo and plan of Agalar Onu Mosque](image)

3.9. Haskoy Mosque

The mosque is located in Haskoy, 12 km from the Finike district of Antalya. The construction date and are unknown. There is no information about the construction date and architect of mosque. The mosque has a square plan covered with a central dome, which sits on an octagonal pulley. The mosque, which is now closed for worship, was restored in 1983 by the General Directorate of Foundations. Photo and plan of Haskoy Mosque shown in Figure 9.

![Figure 9. Photo and plan of Haskoy Mosque](image)
3.10. Takkaci Mustafa Mosque

The mosque is located in Muratpaşa, Antalya. There is no exact information about the construction date and architect of the mosque. The mosque has a square plan covered with a central dome, which sits on an octagonal pulley. Photo and plan of Takkaci Mustafa Mosque shown in Figure 10.

3.11. Haci Hasan Mosque

The mosque is located in Serik, Antalya. According to the mosque inscription; the mosque was constructed in 1820 by Hacı Hasan Ağa. The mosque has a square plan and a main single dome. Photo and plan of Haci Hasan Mosque shown in Figure 11.
3.12. Yesilkaraman Mosque

The mosque is located in yeşilkaraman village, 34 km from Antalya. According to the mosque inscription; the mosque was constructed in 1912 but there is no information about built by whom. The mosque has a square plan and main single dome. Photo and plan of Yesilkaraman Mosque shown in Figure 12.

![Figure 13. Photo and plan of Yesilkaraman Mosque](image)

3.13. Kizilli Mosque

The mosque is located in kızıllı village, 12,5 km from Varsak, Antalya. According to the mosque inscription; the mosque was constructed in 1912 but there is no information about built by whom. The mosque has a square plan and dome which sits on an octagonal pulley. Photo and plan of Kizilli Mosque shown in Figure 13.

![Figure 14. Photo and plan of Kizilli Mosque](image)


The mosque is located in Serik, Antalya. There is no exact information about the construction date and architect of mosque. The mosque has a square plan covered with a central dome. Photo and plan of Alacami Mosque shown in Figure 14.
3.15. Kurus Koyu Mosque

The mosque is located in Kürtüş village, Serik, Antalya. According to inscription; the mosque was constructed in 1930 but there is no information about built by whom. The mosque has a square plan covered with a central dome. Photo and plan of Kurus koyu Mosque shown in Figure 15.

4. Results and Discussions

In this approximate method, it was assumed that the materials were similar of all worship buildings, the volumetric weight of masonry was 20 kN/m³ and β coefficient was equal to 0.037. The values were computed separately for X (longitudinal) and Y (transversal) directions respectively. The values, which exceed threshold, were highlighted with the shaded cells.

Zone A was taken into account for parameter 10. Each parameter has separate threshold value which is computed in accordance with its properties. Three soil type A was used for the application of the approximate method for parameter 10. According to the all average X and Y direction values are usually approximate.
Figure 17. Graph representation of Parameter 1

Figure 18. Graph representation of Parameter 2

Figure 19. Graph representation of Parameter 3 (a)
Figure 20. Graph representation of Parameter 3 (b)

Figure 21. Graph representation of Parameter 4

Figure 22. Graph representation of Parameter 5
Figure 23. Graph representation of Parameter 6

Figure 24. Graph representation of Parameter 7

Figure 25. Graph representation of Parameter 8
Figure 26. Graph representation of Parameter 9

Figure 27. Graph representation of Parameter 10

Figure 28. Graph representation of Parametric Seismic Risk Assessment Method X Direction
In terms of parameter 1, see Table 1, all values of parameter 1 exceed the threshold value except Tekeli Mehmet Pasha Mosque. And the same situation is appropriate for parameter 2, parameter 3, parameter 4, parameter 5, parameter 6, parameter 7 and parameter 9, (see Tables 2 to 11), respectively. In terms of parameter 8 only Tekeli Mehmet Pasha Mosque bellowed threshold value. In terms of values along X and Y direction of parameter- 10 Soil A, all mosque values are below threshold.

**Table 1. The result value of Parameter 1**

<table>
<thead>
<tr>
<th>Worship Buildings</th>
<th>X Direction</th>
<th>Y Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Parameter 1(γ1)</td>
<td>Parameter 1(γ1)</td>
</tr>
<tr>
<td>Suleymaniye Mosque</td>
<td>1.32</td>
<td>1.66</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>1.41</td>
<td>1.38</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>1.08</td>
<td>1.66</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>1.07</td>
<td><strong>0.89</strong></td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>1.38</td>
<td>1.39</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>1.87</td>
<td>1.97</td>
</tr>
<tr>
<td>Muselim Mosque</td>
<td>1.87</td>
<td>1.97</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>2.19</td>
<td>2.58</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>1.45</td>
<td>1.61</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>1.48</td>
<td>1.65</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>2.07</td>
<td>2.23</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>1.65</td>
<td>1.54</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>1.65</td>
<td>1.54</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>1.90</td>
<td>2.12</td>
</tr>
<tr>
<td>Korus Koyu Mosque</td>
<td>1.23</td>
<td>1.66</td>
</tr>
</tbody>
</table>

**Table 2. Result value of Parameter 2**

<table>
<thead>
<tr>
<th>Worship Buildings</th>
<th>X Direction</th>
<th>Y Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Parameter 2(γ2)</td>
<td>Parameter 2(γ2)</td>
</tr>
<tr>
<td>Suleymaniye Mosque</td>
<td>0.63</td>
<td>0.79</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>0.85</td>
<td>0.83</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>0.60</td>
<td>0.92</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>0.80</td>
<td>0.66</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>0.69</td>
<td>0.69</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>0.78</td>
<td>0.82</td>
</tr>
</tbody>
</table>
### Table 3. Result value of Parameter 3 (For Non zero Cohesion)

<table>
<thead>
<tr>
<th>Worship Buildings</th>
<th>X Direction Parameter 3($\gamma_3$)</th>
<th>Y Direction Parameter 3($\gamma_3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suleymaniye Mosque</td>
<td>7.44</td>
<td>9.36</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>9.06</td>
<td>8.84</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>6.80</td>
<td>10.43</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>9.26</td>
<td>7.65</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>8.30</td>
<td>8.33</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>8.55</td>
<td>9.019</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>8.55</td>
<td>9.019</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>10.09</td>
<td>11.86</td>
</tr>
<tr>
<td>Haskoym Mosque</td>
<td>9.71</td>
<td>10.78</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>11.59</td>
<td>12.92</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>13.20</td>
<td>14.25</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>14.77</td>
<td>13.80</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>14.77</td>
<td>13.80</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>12.91</td>
<td>14.41</td>
</tr>
<tr>
<td>Korus Koyu Mosque</td>
<td>13.96</td>
<td>18.92</td>
</tr>
</tbody>
</table>

### Table 4. Result value of Parameter 3 (For zero Cohesion)

<table>
<thead>
<tr>
<th>Worship Buildings</th>
<th>X Direction Parameter 3($\gamma_3$)</th>
<th>Y Direction Parameter 3($\gamma_3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suleymaniye Mosque</td>
<td>4.72</td>
<td>5.94</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>5.39</td>
<td>5.26</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>4.21</td>
<td>6.45</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>5.84</td>
<td>4.82</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>5.32</td>
<td>5.34</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>5.19</td>
<td>5.47</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>5.19</td>
<td>5.47</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>4.90</td>
<td>5.76</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>5.05</td>
<td>5.61</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>5.04</td>
<td>5.62</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>5.13</td>
<td>5.53</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>5.51</td>
<td>5.15</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>5.51</td>
<td>5.15</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>5.04</td>
<td>5.62</td>
</tr>
<tr>
<td>Korus Koyu Mosque</td>
<td>4.52</td>
<td>6.13</td>
</tr>
</tbody>
</table>
Table 5. Result value of Parameter 4

<table>
<thead>
<tr>
<th>Parameter 4</th>
<th>( \gamma_4 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suleymaniye Mosque</td>
<td>0.17</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>0.68</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>0.12</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>0.04</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>0.61</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>0.93</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>0.93</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>0</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>0.85</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>0</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>0.10</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>0</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>0</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>0</td>
</tr>
<tr>
<td>Kurus Koyu Mosque</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 6. Result value of Parameter 5

<table>
<thead>
<tr>
<th>Parameter 5</th>
<th>( \gamma_5 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suleymaniye Mosque</td>
<td>5.71</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>1.68</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>7.76</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>24.00</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>1.87</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>1.22</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>1.22</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>0.00</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>1.34</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>0.00</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>10.67</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>0.00</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>0.00</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>0.00</td>
</tr>
<tr>
<td>Kurus Koyu Mosque</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Table 7. Result value of Parameter 6

<table>
<thead>
<tr>
<th>Worship Buildings</th>
<th>X Direction</th>
<th>Y Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Parameter 6(( \gamma_6 ))</td>
<td>Parameter 6(( \gamma_6 ))</td>
</tr>
<tr>
<td>------------------------------</td>
<td>--------------</td>
<td>--------------</td>
</tr>
<tr>
<td>Suleymaniye Mosque</td>
<td>7.14</td>
<td>7.14</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>7.61</td>
<td>7.61</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>7.89</td>
<td>7.89</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>6.57</td>
<td>6.57</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>4.26</td>
<td>4.26</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>8.19</td>
<td>8.19</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>8.19</td>
<td>8.19</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>11.86</td>
<td>11.86</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>7.37</td>
<td>7.37</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>8.52</td>
<td>8.52</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>13.10</td>
<td>13.10</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>13.44</td>
<td>13.44</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>13.44</td>
<td>13.44</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>12.75</td>
<td>12.75</td>
</tr>
<tr>
<td>Kurus Koyu Mosque</td>
<td>14.67</td>
<td>14.67</td>
</tr>
</tbody>
</table>
Table 8. Result value of Parameter 7

<table>
<thead>
<tr>
<th>Parameter 7</th>
<th>γ_7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suleymaniye Mosque</td>
<td>4.17</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>4.03</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>2.82</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>1.59</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>4.43</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>2.78</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>2.78</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>4.25</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>3.32</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>6.28</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>3.70</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>6.25</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>6.25</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>3.50</td>
</tr>
<tr>
<td>Korus Koyu Mosque</td>
<td>4.11</td>
</tr>
</tbody>
</table>

Table 9. Result value of Parameter 8

<table>
<thead>
<tr>
<th>Parameter 8</th>
<th>γ_8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suleymaniye Mosque</td>
<td>1.70</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>1.70</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>1.31</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>1.15</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>1.32</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>1.65</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>1.65</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>3.83</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>1.76</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>4.27</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>2.55</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>4.77</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>4.77</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>3.24</td>
</tr>
<tr>
<td>Korus Koyu Mosque</td>
<td>5.63</td>
</tr>
</tbody>
</table>

Table 10. Result value of Parameter 9

<table>
<thead>
<tr>
<th>Worship Buildings</th>
<th>X Direction</th>
<th>Y Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Parameter 9(γ_9)</td>
<td>Parameter 9(γ_9)</td>
</tr>
<tr>
<td>Suleymaniye Mosque</td>
<td>0.92</td>
<td>1.01</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>0.99</td>
<td>0.96</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>1.01</td>
<td>1.10</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>1.19</td>
<td>1.00</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>0.90</td>
<td>0.89</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>0.98</td>
<td>0.98</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>1.01</td>
<td>1.06</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>0.94</td>
<td>1.04</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>0.88</td>
<td>0.97</td>
</tr>
<tr>
<td>Worship Buildings</td>
<td>X Direction</td>
<td>Y Direction</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>-------------</td>
<td>-------------</td>
</tr>
<tr>
<td>Suleymaniye Mosque</td>
<td>0.92</td>
<td>1.01</td>
</tr>
<tr>
<td>Bali Bey Mosque</td>
<td>0.99</td>
<td>0.96</td>
</tr>
<tr>
<td>Murat Pasha Mosque</td>
<td>1.01</td>
<td>1.10</td>
</tr>
<tr>
<td>Tekeli Mehmet Pasha Mosque</td>
<td>1.19</td>
<td>1.00</td>
</tr>
<tr>
<td>Omer Pasha Mosque</td>
<td>0.90</td>
<td>0.99</td>
</tr>
<tr>
<td>Nasreddin Mosque</td>
<td>0.98</td>
<td>0.98</td>
</tr>
<tr>
<td>Musellim Mosque</td>
<td>1.01</td>
<td>1.06</td>
</tr>
<tr>
<td>Agalaronu Mosque</td>
<td>0.94</td>
<td>1.04</td>
</tr>
<tr>
<td>Haskoy Mosque</td>
<td>0.88</td>
<td>0.97</td>
</tr>
<tr>
<td>Takkaci Mustafa Mosque</td>
<td>0.74</td>
<td>0.83</td>
</tr>
<tr>
<td>Haci Hasan Mosque</td>
<td>0.97</td>
<td>1.12</td>
</tr>
<tr>
<td>Yesilkaraman Mosque</td>
<td>1.02</td>
<td>1.10</td>
</tr>
<tr>
<td>Kizilli Mosque</td>
<td>0.90</td>
<td>0.84</td>
</tr>
<tr>
<td>Alacami Mosque</td>
<td>0.99</td>
<td>1.10</td>
</tr>
<tr>
<td>Kurus Koyu Mosque</td>
<td>0.79</td>
<td>1.07</td>
</tr>
</tbody>
</table>

In terms of the total parameter calculation results are compared; Suleymaniye mosque, Murat Pasha Mosque, and Kurus Koyu have more risk than other mosques. The risk level of the worship buildings is presented in Figure 30. Risk situation was determined by whether exceeds the parameter values of mosque or not.
5. Conclusion

This paper presents an application of an approximate method for assessment of worship buildings in Antalya. The database includes 15 mosques. These mosques selected according to the availability of information and plan which has one single dome. Ten parameters and thresholds are used. The first six parameters and threshold values are based on Loureiro and Oliveira (Loureiro and Oliveira 2004), so in this study, it was assumed that threshold values of the first six parameters to be equally applicable for the worship buildings in Antalya.

Generally, the X and Y direction of the worship buildings values are approximately each other. It is thought that the reason for these approximate values is buildings plan, which has square and symmetrical. In terms of the average results all parameters have acceptable results, according to the total parameter formulate results Murat pasha mosque, Korus koyu mosque, and Suleymaniye mosque are high risky than other worship buildings, so that can be said that other mosques are more reliable under seismic risk.

The methods and parameters as indicators for fast screening and decision to prioritize deeper studies in historical masonry buildings and to assess vulnerability to seismicity. In general, the values of the directions, which are longitudinal (y) and transversal (x), are approximate. The analysis of the parameters shows that a logical common trend can be established. It is very difficult to determine how a masonry building responds against seismic loads. In this regard, there should make seismic analysis by using analytical and experimental methods. Many historical masonry buildings are protected by the General Directorate of Foundations because of their cultural values. Therefore, the examination of buildings in many aspects involves a challenging process. The seismic assessment of the structures should not cause any damage. In this process, it is thought that the parametric seismic evaluation method, which is made considering the geometric and some structural features of the structures and gives approximate results, will meet the need in the first stage.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Photosynthetic Microbial Desalination Cell to Treat Oily Wastewater Using Microalgae *Chlorella Vulgaris*

Suhad S. Jaroo a,*, Ghufran F. Jumaah a, Talib R. Abbas b

a Civil Engineering Department, University of Technology, Baghdad, Iraq.

b Environment and Water Directorate, Ministry of Science and Technology, Baghdad, Iraq.

Received 25 June 2019; Accepted 20 September 2019

Abstract

Microbial desalination cell (MDC) offers a new and sustainable approach to desalinate saltwater by directly utilizing the electrical power generated by bacteria during organic matter oxidation. In this study, we used microalgae *Chlorella Vulgaris* in the cathode chamber to produce oxygen as an electron accepter by photosynthesis process for generate bioelectricity power and treat oil refinery wastewater by microorganisms in both anode and cathode.

The power density generated by this Photosynthetic Microbial Desalination Cell (PMDC) with 1KΩ external resistance at the first 4th hr. of operation period was 0.678 W/m³ of anode volume and 0.63 W/m³ of cathode volume. It increased after one day to a peak value of (4.32 W/m³ of anode volume and 4.013 W/m³ of cathode volume). The microalgae growth in the biocathode chamber followed in terms of optical density. The optical density increased from 0.546 at the beginning of the system operation to 1.71 after 24 days of operation period. The percentage removal of chemical oxygen demand (COD) of oil refinery wastewater was 97.33% and 79.22% in anode and cathode chamber, respectively. The microalgae in the biocathode were able to remove volatile compounds causing odor from the influent wastewater. TDS removal rate 159.722 ppm/h with initial TDS in desalination chamber of 35000 ppm.

Keywords: PMDC; Oil Refinery; *Chlorella Vulgaris*.

1. Introduction

Industrial wastewater generated from the oil industry generally characterized by its high concentration of pollutants such as organic compounds, heavy metals, and chemicals, which may cause adverse public health and environmental problems [1]. Conventional techniques (chemical precipitation, membrane filtration, electrolytic processes, and adsorption) have widely used for the treatment of such wastewater. However, these techniques present many disadvantages, such as high cost, intensive energy requirements, and considerable sludge generation [2]. Moreover, wastewater treatment and reuse have become an essential issue with the increasing population and depletion of freshwater resources in many regions of the world.

Microbial fuel cell (MFC) is a promising technology that has obtained a significant interest in recent years. This technology offers the possibility of treating a wide range of wastewaters with soluble organic pollutants and gaining electrical current simultaneously [3]. MFC technology based on the electrogenic nature of specific bacteria that use electrode (anode) as an electron acceptor instead of dissolved oxygen while treating wastewater anaerobically. The electron transferred to the cathode through an external electric circuit at which the reduction reaction occurs [4].

*Corresponding author: 42102@student.uotechnology.edu.iq

http://dx.doi.org/10.28991/cej-2019-03091441

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
Microbial desalination cell (MDC), consists typically of an anode, desalination and cathode chambers, is an amendment of MFC which featured with wastewater treatment, desalination (brackish water and seawater), and bioelectricity production simultaneously [5]. However, MFC and MDC also suffer from low power densities due to power losses in electron transfer and release mechanisms, especially in the cathode chamber. To the performance of the MFC improves, cathodes often painted with noble catalysts such as platinum in case of air-cathode MDC with external aeration provided or using chemical agents such as ferricyanide to achieve faster redox kinetics [6-9]. High cost and toxicity problems when using noble catalysts and chemical electrolytes. So, to avoid that, biocathodes can be used as an alternative to abiotic cathodes [10-14]. Another advantage of using different biocathodes is that the active microbial metabolism can be used to produce beneficial products [15] or eliminate nutrients from wastewaters, such as nitrate and contaminants such as heavy metals [16, 17]. Different microbial populations can be used as biocatalysts in biocathodes such as nitrifying and denitrifying bacteria or microalgae to produce electron acceptors required for reduction reaction at the cathode [18, 19]. Microalgae Chlorella Vulgaris was utilized by many studies in bio cathode microbial desalination cells to generate oxygen (electron acceptor) by the photosynthetic process. For example, Bahareh and Veera [10] used microalgae C. Vulgaris in biocathode photosynthetic microbial desalination cell (PMDC) for clean energy, water, and biomass production. Thomas and Veera [20] used sodium bicarbonate as an inorganic carbon source for microalgae C. Vulgaris in biocathode PMDC and studied its effects on the system performance. In addition to oxygen generation microalgae such as Chlorella sp. can achieve elimination nitrogen, phosphorus, CO₂, and toxic metals from various types of wastewaters, making them an attractive alternative for wastewater treatment [21, 22].

Many studies investigated the ability of microalgae Chlorella Vulgaris to treat different types of wastewater. For example, Madadi et al. [23] studied the biological treatment of petrochemical wastewaters by microalgae Chlorella Vulgaris. The results of this study showed that a combination of surfactants and Chlorella Vulgaris is an efficient approach for pre-treatment of wastewaters and can use for the removal of nutrients from petrochemical wastewaters. Microalgae biomass harvesting from microalgae bioreactor treating textile wastewater studied by Hala and Laila [24]. This work revealed the adaptability of the microalgae C. Vulgaris in textile wastewater and its ability to mitigate the waste effluent by elimination color and Chemical Oxygen Demand (COD) and the ability of the bioreactor to produce biomass. Liang Wang et al. [25] studied the cultivation of green algae Chlorella sp. in various wastewaters from municipal wastewater treatment plants and their ability to remove nitrogen, phosphorus, COD, and heavy metals from the wastewaters. Luz T. et al. worked to improve a procedure for biological treatment of wastewater from ethanol and citric acid production industry using the microalgae Chlorella Vulgaris and the macrophyte Lemna miniscule [26]. Biocathode microbial fuel cell was used by Ronald et al. [27] for biotreatment of oils and fats from wastewater of a chocolate factory in Chlorella Vulgaris microalgal cathode chamber and removal of blue dye brl in the anode chamber of the MFC by the bacterial community.

This study aimed 1st to use microalgae C. Vulgaris in the cathode chamber as an oxygen generator and 2nd to treat oil refinery wastewater in both anode and cathode chambers of a PMDC and to investigate its performance in terms of wastewater treatment, salty water desalination, and bioelectricity generation. PMDC performance was evaluated in terms of COD removal percent, desalination rate, and electricity production. The maximum power densities profiles and desalination rates were derived from the experimental results.

2. Materials and Methods

2.1. PMDC Configuration

The PMDC used in this work consisted of three chambers (anode, desalination, and cathode) of plexiglass with an internal cross-section area 10x10 cm and length of 7, 3, 7 cm for the anode, desalination, and cathode chamber, respectively as described in (Figure 1-A). Cation exchange membrane (CEM, CMI 7000, Membranes international) separated the cathode and the desalination chambers while an anion exchange membrane (AEM, AMI 7001, Membranes international) separated the anode and the desalination chambers. Both membranes were preconditioned by submerging in a 5% NaCl solution for 24 h and rinsed with deionized water before use, to allow for membrane hydration and expansion as recommended by the supplier. Graphite plate electrodes were used in anode and cathode chambers with dimensions 9x9 cm and thickness 2mm. The electrodes were installed at a distance of 1 cm from the respective ion exchange membranes. Before use, the electrodes were drenched in deionized water for a period of 24 hr. The copper wires were used to establish contact between the electrodes, and these contacts were sealed with epoxy.

Each of the three chambers has two ports (inlet and outlet) with 1cm diameter except the inlet port in the cathode chamber with 3cm diameter suitable for dissolved oxygen (DO) measurement by a portable meter. The anode chamber was closed with a screw to ensure an anaerobic microenvironment. The working volume of anode, desalination, and cathode chambers were 650, 300, 700 mL, respectively. Illumination on the algae cathode chamber was provided by white light at 23W. The PMDC illustrated in (Figure 1-B)
2.2. Inoculation of microorganisms in PMDC

The experimental work in this study comprised of three stages, 1st and 2nd stages for two purposes:

1. Acclimation PMDC (biofilm creation on electrodes from anaerobic bacteria in anode and microalgae C. Vulgaris in cathode)

2. Test the ability of system for cultivation biomass of microalgae.

In the 3rd stage, the PMDC used to treat oil refinery wastewater in both anode and cathode, all three stages illustrated in (Figure 3.). So, 1st and 2nd stages have the flowing:

1. Microbial consortium

The microbial consortium in the anode chamber was collected from the activated sludge basin of the wastewater treatment plant of Al-Doura refinery in Baghdad city. The sludge was allowed to acclimatize to anaerobic conditions in synthetic wastewater containing 500 mg/L of COD for 24 days. The acclimation process is done in the MDC to enhance the formation of the biofilm layer on the anode. The synthetic wastewater was consisted of: 500 (mg/l) of sodium acetate in buffer solution of KH$_2$PO$_4$ (40 mg/l), NH$_4$Cl (200 mg/l), CaCl$_2$ (40 mg/l), MgCl$_2$ (40 mg/l), KCl (40 mg/l) and 10 ml of trace mineral solution.
2. Microalgae C. Vulgaris

The microalgae Chlorella Vulgaris (Suncoast marine aquaculture, SCMAlabs.com) used in the cathode compartment was examined by microscope (Figure 2.) and was grown in the following mineral solution for 24 days to cultivation algae and form biofilm in cathode chamber: CaCl2 (0.025 g/l), NaCl (0.025 g/l), NaNO3 (0.25 g/l), MgSO4 (0.075 g/l), KH2PO4 (0.105 g/l), K2HPO4.3H2O (0.075 g/l), and 3 ml of trace metal solution with the following concentration was added to 1000 mL of the above solution: FeCl3 (0.194 g/l), MnCl2 (0.082 g/l), CoCl2 (0.16 g/l), Na2MoO4.2H2O (0.008 g/l), and ZnCl2 (0.005 g/l) [11] with buffer solution: KH2PO4 (2.25 g/l), K2HPO4.3H2O (2 g/l), NH4Cl (0.31 g/l) and KCl (0.13 g/l) and NaHCO3 (500 mg/l) as an inorganic CO2 source [20].

Figure 2. Micrograph of microalgae C. Vulgaris using an optical microscope

3. Saltwater

Saltwater in the medial chamber had a different concentration of NaCl (15 g/l) for the 1st stage and (35 g/l) for the 2nd stage to investigate different behaviours of PMDC with varying concentrations of salt.

After 1st and 2nd stages (acclimation period), 3rd stage started (application period). In this stage, actual wastewater treated in both anode and cathode chamber. The actual wastewater was the effluent of dissolved air flotation (DAF) unit in the wastewater treatment plant of Al-Doura refinery in Baghdad city with COD (75 mg/l) and oil content of (28.1 mg/l). The wastewater was added to anode and cathode at the same time. In the cathode chamber, wastewater was mixed with 10% of algae solution. Salt concentration in the desalination chamber for this stage was (35 g/l).

Figure 3. Flow chart of the operation stages of PMDC

2.3. Analyses and Calculations

The voltage was recorded using a digital multimeter (Aswar, DT860D) with a 500 Ω external resistor at the acclimation period (1st and 2nd stages) and 1KΩ at the application period (3rd stage) connecting the anode and the cathode was used in closed circuit tests. The current was determined using Ohm's law, I = Voltage/R. The power density was
calculated as per the anode/cathode chamber volume or the electrode surface area. Coulombic efficiency (CE), defined as the partial recovery of electrons from the substrate, was calculated by using Equation 1 [28]:

$$CE = \frac{M \int_0^t I dt}{nFV_a (COD_0 - COD_t)} \times 100$$  \hspace{1cm} (1)

Where (M) is the molecular weight of oxygen, (I) is the current, (F) is Faraday’s constant, n = 4 is the number of electrons exchanged per mole of oxygen, and V_a is the anolyte volume. COD_0 represents the influent wastewater chemical oxygen demand to the anode chamber, and COD_t is the COD value after the time (t).

After attaining steady-state operation, polarization curves were obtained by changing the external resistance from 10 kΩ to 100 Ω in steps (about 15 min for each step to reach a steady-state). COD tests were executed using standard methods [29]. Electrical conductivity, TDS removal, pH of the samples and DO in the cathode chamber were recorded using a conductivity meter, pH meter and dissolve oxygen meter with temperature meter by the same multimeter (Lovi Bond Senso Direct 150), respectively. The desalination rate was calculated using Equation 2:

$$TDS_{\text{removal rate}} (\text{ppm/h}) = \frac{C_0 - C_t}{t}$$  \hspace{1cm} (2)

Where, C_0 and C_t are the initial and the final TDS of saltwater in the desalination chamber over a batch cycle of time t.

Microalgae growth was observed by measuring the optical density of the microalgae suspension with a UV 2300 spectrophotometer (Techcomp, Spain) at a wavelength of 620 nm [10].

### 3. Results and Discussion

#### 3.1. Power Production

The voltage generation between anode and cathode in the 1st and 2nd stages with two operation cycles to every stage were illustrated in Figure 4. (A and B). At the beginning of each cycle in the 1st stage the anode chamber was fed with influent wastewater with 500 mg/l of COD, the desalination chamber was filled with salty water with 15 g/l of TDS, and the cathode chamber was fed with microalgae. The absorbance for microalgae suspension was 0.546. While, 500 mg/l of COD, 35 g/l of TDS was used at the beginning of the 2nd stage. The absorbance for microalgae suspension at the beginning of the 2nd stage (resulted from the 1st stage) was 1.061. No additional algae suspension was added. It was clear that the maximum voltage difference across the external 500Ω electrical resistance between anode and cathode was 72 mV and 45 mV (1st stage), and 127 mV and 83 mV (2nd stage) for cycles 1 and 2, respectively; during six days for each cycle. The maximum voltage with open circuit mode varied between 581 mV and 568 mV (1st stage) and 706 mV and 597 mV (2nd stage) for cycles 1 and 2, respectively.

It should be noted that the electricity generation activity has decreased in the second cycle for the two stages which might be attributed to the decreasing of conductivity in the middle chamber, so the ions transferring between compartments has declined and decreased in power generation activity. At the same time, the electricity generation activity has improved in the 2nd stage comparable with that in the 1st stage, as evidenced by the values of maximum voltage as shown in (Figure 4). That is due to the biofilm formation on the electrode in anode chamber and increase in biomass of algae in cathode chamber (increasing in oxygen generation) in addition to the increase of conductivity in the middle chamber that improved ion exchange process between compartments and rising power generation activity.
3.2. Microalgal Growth of C. Vulgaris on the Cathode

The growth of microalgae C. Vulgaris monitored by measuring the optical density at the beginning of the acclimation period, at the end of the 1st stage (with 15g/l of TDS) and the end of the 2nd stage (with 35g/l of TDS). The optical density was 0.546, 1.061 and 1.71 at the beginning, end of the 1st stage, and end of 2nd stage, respectively. These values are shown in (Figure 5.). The growth of algae in this study was higher than that in the previous study [10] the algae growth presented as optical density increased from 0.401 to 0.63 due to the difference of catholyte volume and in this study was used sodium bicarbonate as an inorganic carbon source that enhanced algae growth. That meaning this system was active in the cultivation of algae that can be used as biofuel.

3.3. Polarization Analysis

After the acclimation period, the PMDC was used to treat oil refinery wastewater in both anode and cathode compartment. After many cycles of operation in this mode, the power production from actual wastewater treatment was illustrated in the polarization curve shown in (Figure 6). The operation conditions were as follows: 75 mg/l of COD (anode chamber), 35 g/l of TDS (desalination chamber), 10% for microalgae suspension to wastewater in biocathode chamber, light is on the maximum power density was 277.8 mW/m² with maximum current density of 18.52 mA/m².
Figure 6. A - Voltage values with multi-resistance, B - Polarization curve, and C - power density versus current density.
Figure 7. Values of the power density concerning anode volume (A) and cathode volume (B) at 1KΩ resistance with COD values

For a typical cycle, the power density produced with 1KΩ external resistance between anode and cathode at first 4th hr of the run period was 0.678 W/m³ of anode volume and 0.63 W/m³ of cathode volume. It increased after one day to the peak (4.32 W/m³ of anode volume and 4.013 W/m³ of cathode volume) are shown in (Figure 7). The peak values of power density in this study more than values that were presented by previous studies for PMFC [27] used algae in biocathode to treat actual wastewater which might be attributed to different system configuration.

3.4. Performance of PMDC to Remove COD

The percentage of COD removal of oil refinery wastewater by anode and cathode was 97.33% and 79.22%, respectively during the cycle of three days as showed in (Figure 8-A). That is evidence of the good ability of microalgae C. Vulgaris in biocathode to treat this kind of wastewater. So, in addition to using microalgae in biocathode as a source of oxygen by the photosynthetic process, it can be used to treat oil refinery wastewater also. Therefore, the PMDC can be used to treat wastewater in both anode and cathode for more sustainability. The values of COD reduction shown in (Figure 7). Our observations noticed the diminishing of oil odor from cathode effluent. This observation was not seen in the anode chamber, although it treated the same wastewater. Therefore, the algae might be able to adsorption the volatile compounds responsible for the odor in oil refinery wastewater. Previous studies used algae to treat multi kinds of industrial wastewater with different percentage of COD removal for every case. The study [25] used algae C. Vulgaris in municipal wastewater treatment plant (MWW) for cultivation algae, the maximum COD removal percent 83% for MWW by algae. The study [28] used algae C. Vulgaris in biocathode MFC for producing electrical energy and treat oils and fats of chocolate factory wastewater, the COD removal percent (cathode 78.6%). The algae C. Vulgaris was used for bioremediation of textile waste effluent in the previous study [24] and COD removal percent 17.5%. Treatment of
Petrochemical Wastewater by algae *C. Vulgaris* during the last study [23] the maximum COD removal was 38%. By comparing between this study and previous studies COD removal percent in this study butter than other studies except for the research of using microalgae for treating MWW [25].

The values of COD reduction in the anode with columbic efficiency (CE) values versus time illustrated in (Figure 8-B). The maximum values of CE were 1.244%, and this value lower than the value presented by the previous study [11] (17.2%). That might be attributed to the big difference between the initial value of COD in both studies (75 mg/l for the present study and more than 1000 mg/l for the previous study).

![Figure 8. A) COD removal percent of anode and cathode and B) the values of COD and CE. Versus time](image)

**3.5. TDS Removal Rates During the Operation of the Different Cases**

For different stages of operation, different TDS removal rates achieved. They were 22.153, 65.625, 159.722 ppm/h for 1\textsuperscript{st}, 2\textsuperscript{nd}, and 3\textsuperscript{rd} stage with initial TDS in desalination chamber 15000, 35000, 35000 ppm, respectively as illustrated in (Figure 9-A). Regarding the 1\textsuperscript{st} and 2\textsuperscript{nd} stages, the difference between the concentrations of salt in the desalination chamber should cause variation in the TDS removal rate due to ion diffusion and osmotic process. In (Figure 9-B), the initial conductivity of anode and cathode chambers for 2\textsuperscript{nd} stage were 2.2 and 10.43 mS/cm, respectively. On the other hand, the initial conductivity of anode and cathode for 3\textsuperscript{rd} stage 1.433 and 1.993 mS/cm, respectively, which explained the significant difference of the TDS removal rate between these two stages for the same initial salt concentration in the desalination chamber. The lower conductivity in anode and cathode chambers should improve the ability of ions transfer outside the desalination chamber. The changing of pH values of cathode and anode in the 3\textsuperscript{rd} stage from initial value approximately 7 in both anode and cathode to 6 for the anode and 9 for the cathode was due to the desalination process were shown in (Figure 9-C). The pH in the anode chamber reduced due to a cumulation of H\textsuperscript{+} ions produced from the organic material biodegradation process by bacteria in the anode chamber. The AEM has prevented H\textsuperscript{+} ions from transporting to the desalination chamber; instead, it was allowed chloride ions transferring from the desalination chamber.
to the anode chamber. On the other hand, the pH in the cathode chamber increased due to the depletion of \( \text{H}^+ \) ions due to stimulating them with oxygen. CEM prevented \( \text{OH}^- \) ion to transport to desalination chamber; instead, it was allowed sodium ions transferring from desalination chamber to the cathode chamber.

**Figure 9.** A) The TDS removal rate for the three stages, B) the conductivity values in the anode and cathode chambers for 2\textsuperscript{nd} and 3\textsuperscript{rd} stages (the run period for 2\textsuperscript{nd} stage 12 days and 3\textsuperscript{rd} stage 3 days), and C) pH values of anode and cathode during the 3\textsuperscript{rd} stage

---

2695
### 3.6. Dissolve Oxygen Production by Algae C. Vulgaris in Biocathode PMDC

Dissolve oxygen production by the photosynthetic process of algae in biocathode during the illumination period (12h) is illustrated in (Figure 10-A) by initial, highest, and final values of DO for 1st stage, 2nd stage, and 3rd stage. DO was varied between (6.8-4), (8.5-6.1) and (7.1-5.5) for 1st stage, 2nd stage, and 3rd stage, respectively. It is worth noting the higher DO production in 2nd stage might be due to an increase in the biomass of algae represented by optical density and biofilm formation inside the biocathode chamber. DO values in the 3rd stage lower than 2nd stage because of the quantity of algae solution so tiny just 10% of cathode volume and that values of DO produced from the biofilm layer inside the biocathode chamber. At the end of every stage, the value of DO decreased. That might be due to the declining of the substrate in cathode that algae feeding on it. In comparison with previous studies [10, 11], DO concentrations in this work were approximately the same values presented by these studies.

Dissolve oxygen production should control the power generation because it acts as an electron acceptor required for completing the redox reaction in the PMDC. (Figure 10-B) Shows the variation of power generation between light and dark during the operation period (12 to 12 light and dark cycle) for treatment oil refinery wastewater (3rd stage) in anode and cathode by measuring the voltage and dissolve oxygen every four hours. The effect of illumination on the DO production and power generation was apparent, as shown in (Figure 10-B). DO, and voltage was improved during the light period and declined during the dark period. For the first day, the highest amount of voltage (53 mV) with the highest value of DO production (7.6 mg/l) and after that the voltage decline to (25 mV) with DO reduction to (4.8 mg/l).

![Figure 10. A) Initial, highest and final values of DO for three stages and B) The variation of DO and voltage between light and dark period for the 3rd stage](image-url)
4. Conclusion

In this work a PMDC in which microalgae were used in the cathode chamber to produce oxygen by photosynthesis process instead of an air pump or chemical materials like ferricyanide in air cathode MDC and this type called photosynthesis biocathode MDC. The peak of power density generated by this Photosynthetic Microbial Desalination Cell (PMDC) with 1KΩ resistance was (4.32 W/m³ of anode volume and 4.013 W/m³ of cathode volume). The microalgae growth in the biocathode chamber presented by the optical density was increased from 0.546 at the beginning of the system operation to 1.71 after 24 days of operation period. The percentage removal of chemical oxygen demand (COD) of oil refinery wastewater was 97.33% and 79.22% in anode and cathode chamber, respectively. TDS removal rate 159.722 ppm/h with initial TDS in desalination chamber of 3500 ppm. This study proved the possibility of use PMDC for treatment actual oil refinery wastewater in both anode and cathode chambers by using different microorganisms, anaerobic bacteria to treat the wastewater in the anode chamber and microalgae Chlorella Vulgaris to treat the wastewater in the cathode chamber. This method is featured with sustainability because it provides many things at the same time with environment-friendly manner:

- Treat actual oil refinery wastewater in anode and cathode chambers.
- Produce oxygen as an electron accepter by photosynthesis process of microalgae without any bad residual in spite of chemical materials are used in air cathode MDC.
- Generate electricity power.
- Improve the growth of microalgae in the cathode chamber and that use to generate microalgae that can be used to produce biofuel.

5. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Selection of the Optimal FBG Length for Use in Stress-Strain State Diagnostic Systems

Viktor M. Vikulov a*, Aleksandr V. Todorov a, Alexey V. Faustov b, Nikolay L. Lvov b

a Scientific and Innovation Center, Institute for the Development of Research, Development and Technology Transfer, Moscow, Russia.
b Ph.D., Scientific and Innovation Center, Institute for the Development of Research, Development and Technology Transfer, Moscow, Russia.

Received 27 August 2019; Accepted 16 November 2019

Abstract

The article discusses fiber-optic sensors (FOS) based on the Bragg gratings for measuring systems for diagnostics of stress-strain state. Currently, such diagnostic systems are widely used in construction, industry and civil engineering. The physical principle of deformation diagnostics using FOS. The issues of mounting the sensor on the measured area (detail) are separately discussed. The principle of processing the hardware and software of sensors based on Bragg gratings is described. Research method - bench experiments that were carried out on an equal-deformation beam in order to evaluate the change in the width of the reflected FOS peak at different lengths recorded by the Bragg gratings in order to determine the optimal one. The change in the spectrum of the reflected peak under various deforming influences was monitored. Based on the results obtained, recommendations are made on the use of gratings of various lengths in the diagnostic systems for the stress-strain state of parts and assemblies for civil engineering tasks.

Keywords: Bragg Grating; FBG; FOS; Deformation; Spectrum; Optical Signal.

1. Introduction

Currently, sensors on fiber Bragg gratings (FBG) are widely used to monitor various external influences on the structures of buildings and structures, aircraft structures, etc. [1]. The principle of operation of the FBG is based on the back reflection of the light passing through it at the Bragg wavelength, which depends on the period of the recorded lattice and the effective refractive index [2-4]. Both of these parameters are linearly dependent on changes in temperature and strain that affect the fiber. Tracking in the spectral region of the Bragg wavelength data of the lattice makes it possible to implement sensors of external influences such as temperature and strain gauges [5]. FBG is schematically represented in Figure 1.

Measurement of deformations and temperature at critical points of various structures, both building and aviation, is an important element in monitoring their condition [6, 7]. The use of sensors based on FBG in this case has a number of advantages, such as:

- Small dimensions;
- Lack of sensitivity to electromagnetic interference;
- High resistance to aggressive environments;
- High sensitivity.

*Corresponding author: v_vikulov@nicirt.ru

http://dx.doi.org/10.28991/cej-2019-03091442

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
To develop a measurement system based on FBG, it is necessary to analyze the deviations of the parameters of various sensor manufacturers for changes in the reflected peak parameters, which can affect the accuracy of measuring external influences using FBG. One of the main parameters in this case is the peak width, because when using compact spectrometers such as BaySpec [8] or Ibsen I-MON [8], which have a resolution of 512 points in the spectral range from ~ 30 to ~ 70 nm, to the reflected the Bragg peak is only up to 3-5 points per peak. The purpose of the experiments described below was to determine the optimal FBG parameters for developing systems for measuring the stress-strain state.

This article discusses FBGs of two manufacturers made using the same technology of recording with a phase mask [9-11], in standard germanosilicate fibers of the SMF-28 type, except for fibers with increased mode retention due to a reduced core and a complex refractive index profile. The relevance of this study is due to the large number of publications related to the description of systems for diagnosing the stress-strain state of various parts and assemblies (structures, machines, movable equipment) based on FBG [12, 13].

2. Stand Description

The main objective of the tests is to compare changes in the central wavelength of the reflected signal from different (FBG) with the same deforming effect on the mounting detail of the FBG. The tests were carried out at room temperature and at elevated (+80 °C).

Types of FBG used in the tests:

- Type 1, Manufacturer No. 1, the length of the recorded lattice is 10 mm, the fiber type is analogue to SMF-28 (designation on the graphs D1530_CH_10).
- Type 2, Manufacturer No. 2, the length of the recorded lattice is 10 mm, the fiber type is analogue to SMF-28 (designation on the graphs D1535_IT_10).
- Type 3, Manufacturer No. 2, the recorded lattice length is 3 mm, the fiber type is analogue to SMF-28 (designation on graphs D1560_IT_03).
- Type 4, Manufacturer No. 2, the recorded lattice length is 3 mm, the type of fiber is fiber with increased mode retention (designation on the graphs D1545_IT_10_TC).

The reflected peaks of FBGs of Type 3 and 4 with a grating length of 3 mm are approximately 2 times wider than the peaks of FBG with a length of 10 mm, which is usually considered a negative factor. However, in the case of low-resolution spectrometers, wider spectra may be advantageous since they will have a larger number of dots per peak. The parameters of the reflected Bragg peak were analyzed at the experimental bench, see Figure 2. The Yokogawa AQ6370 C optical spectral analyzer was used as a spectrometer.

The full functional diagram of the stand is shown in Figure 2.
For measurements, FOS were mounted on the surface of an equal strain plate using Araldite © glue [14]. A view of an equal strain plate with sensors on a load tool with a micrometer screw is shown in Figures 3 and 4, respectively.

After connecting the measuring stand using a micrometer screw, deformation was applied to the beam with a pitch of 2.5 mm; for thermal tests, the step was 5 mm. The data obtained from the spectrum analyzer were entered in the table. Two values were shot:

- Indications of the FBG wavelength at a given bias, i.e. wavelength at the peak of the spectrum of the signal from the FBG.
- The width of the spectrum of the signal from the FBG (at a level of 3 dB from the peak).

For a more accurate comparison, the wavelengths were converted to relative units (normalized) by the formula: \( \lambda_t = \frac{\lambda_f - \lambda_0}{\lambda_0} \).

Figure 5 shows an example of the spectrum of the return signal from the FBG.
When the peak position deviated due to an increase in the applied strain, the spectrum width of the reflected Bragg peak was analyzed [15], which directly shows the quality of the return signal.

4. Test Results

Figures 6 to 9 show the dependency graphs for room and elevated temperatures during compression and tension of the plate.

Figure 5. Spectrum of the return signal from FBG

Figure 6. Change in the width of the spectrum of the optical signal with increasing external exposure, compression at a temperature of +25 °C
Figure 9. Change in the spectrum width of the optical signal with increasing external exposure, compression at a temperature of +25 °C

Figure 8. Changing the width of the spectrum of the optical signal during compression. Temperature +80 °C

Figure 9. Changing the width of the spectrum of the optical signal in tension. Temperature +80 °C
5. Practical Findings

As a result of the tests, the following practical conclusions can be drawn:

1. All tested FBGs show equally stable values of the change in deformation under normal conditions (room temperature). However, the spectra of the signals reflected from the FBG undergo significant changes. So, Type 1 and Type 2 sensors indicate the occurrence of side peaks near the main one. The second peaks tend to move in the vicinity of the main peak with an increase in the deforming effect. An example of the appearance of peaks is shown in Figure 10.

![Graph](a)

![Graph](b)

Figure 10. The occurrence of second peaks during deformation on the sensors Type No. 1 (a) and Type No. 2 (b)

With a large number of spectrum points during processing, this problem is not significant, since the peaks are algorithmically distinguishable and can be filtered by setting a threshold intensity value.

In the case of the compact spectrometers described above, in which the resolution is much lower than the laboratory position of the peak, it is often necessary to determine only three points. Thus, the appearance of significant side peaks can lead to an increase in the error in accurately determining the position of the peak.

2. When heated, the deformation characteristics shown by the FBG had significant differences, which can be explained by the thermal expansion of the glue and sensors. The worst characteristics of a decrease in intensity and a departure from the reference type of spectrum were shown by Type No. 4 (Figure 11), which is probably due to the imperfection of recording technology in fibers with a complex shell structure.
Figure 11. Changes in the shape of the spectrum of FBG Type No. 4

A Type 3 sensor, having a length of 3 mm, showed the best spectral characteristics. The spectrum view of this FBG did not change its appearance at all load values and with temperature changes (Figure 12).

Figure 12. Spectrum of FBG Type No. 3. At room temperature without deformation (a), at +80°C with maximum deformation (b)

This advantage is due to the localization of the sensor on a smaller area, which leads to less susceptibility to mounting errors and possible heterogeneities in the adhesion of glue to the fiber.
6. Conclusion

Based on the results obtained, it can be concluded that for further testing and application in strain-measuring systems based on FBG, it is better to use shorter gratings, since they have the most stable spectral characteristics. In addition, the shorter grating length, leading to broadening of the spectrum, is also an advantage compared to longer gratings with a narrow spectrum when used in measurement systems using compact low-resolution spectrometers. These results will help to significantly improve and stabilize FBG diagnostic systems.

In the future, it makes sense to test the FBG with the shortest possible length (1 mm for ITMO University). This type of FBG can be promising for introduction into composite materials (CM), since the localization of the deformation effect will be minimal. This characteristic is important because of the features of the technology for the production of parts from CM - baking and pressure. With a smaller base of the sensitive part of the FBG, the likelihood of unwanted compression and, as a result, changes in the spectral characteristics of the FOS embedded in the CM decreases.

7. Funding

The authors are grateful to the Russian Ministry of Education and Science for co-financing a project to develop a system for detecting stall phenomena on the rotor blades of a helicopter during flight and monitoring the technical condition of the swashplate (agreement No. 14.579.21.0150, identification number RFMEFI57917X0150), within the framework of which results were obtained that were included in this article.

8. Conflicts of Interest

The authors declare no conflict of interest.

9. References

Finite Element Analysis of Beam – Column Joints Reinforced with GFRP Reinforcements

Balamuralikrishnan R. a*, Saravanan J. b

aAssistant Professor, Department of Civil and Environmental Engineering, College of Engineering, National University of Science and Technology, Muscat, Sultanate of Oman.
bAssociate Professor, Department of Civil and Structural Engineering, Annamalai University, Pin:608001, Tamilnadu, India.

Received 04 July 2019; Accepted 17 October 2019

Abstract
Glass Fibre Reinforcement Polymer (GFRP) reinforcements are currently used as internal reinforcements for all flexural members due to their resistance to corrosion, high strength to weight ratios, the ability to handle easily and better fatigue performance under repeated loading conditions. Further, these GFRP reinforcements prove to be the better alternative to conventional reinforcements. The design methodology for flexural components has already come in the form of codal specifications. But the design code has not been specified for beam-column joints reinforced internally with GFRP reinforcements. The present study is aimed to assess the behaviour of exterior beam-column joint reinforced internally with GFRP reinforcements numerically using the ABAQUS software for different properties of materials, loading and support conditions. The mechanical properties of these reinforcements are well documented and are utilized for modelling analysis. Although plenty of literature is available for predicting the joint shear strength of beam-column joints reinforced with conventional reinforcements numerically, but no such study is carried for GFRP reinforced beam-columns joints. As an attempt, modelling of beam-column joint with steel and with GFRP rebars is carried out using ABAQUS software. The behaviour of joints under monotonically increasing static and cyclic load conditions. Interpretation of all analytical findings with results obtained from experiments. The analysis and design of beam-column joints reinforced with GFRP reinforcements are carried out by strut and tie model. Strut and Tie models are based on the models for the steel reinforced beam-column joints. The resulting strut and tie model developed for the GFRP reinforced beam-column joints predicts joint shear strength. Joint shear strength values obtained from the experiments are compared with the analytical results for both the beam-column joints reinforced with steel and GFRP reinforcements. The joint shear strength predicted by the analytical tool ABAQUS is also validated with experimental results.

Keywords: Reinforced Concrete; Beam-column Joint; GFRP Reinforcements; Failure Mechanism; ABAQUS; Strut and Tie Model.

1. Introduction

Recent developments in concrete composites have resulted in several new products which aim to improve the strength, stability and Serviceability of the concrete structures. In general, the structural system comprises of structural elements (load-carrying, such as beams and columns) and non-structural elements (such as partitions, false ceilings, doors). The function of the structural elements is to resist efficiently the action of gravity and environmental loads, and the serviceability of the structure without significantly disturbing the geometry and integrity. In the viewpoint of simplified analysis as one-dimensional skeletal elements such as beams, columns, arches, truss elements or two-

*Corresponding author: balamuralikrishnan@nu.edu.om
http://dx.doi.org/10.28991/cej-2019-03091443
© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
dimensional elements such as slabs, plates and shells are considered. A few structural elements such as beam-column junctions, perforated shear walls may require more rigorous analysis.

In many such instances, where the behaviour of system cannot be predicted, a useful procedure is to idealise the member or region as a series of reinforcing steel tensile ties and concrete compressive struts, interconnected at nodes to form a truss capable of transmitting the loads to the supports, and detailing the reinforcement accordingly. This *strut-and-tie* concept is depicted. It is a very basic concept in structural design that, for transferring a system of loads to the supports, any stable skeletal framework such as a truss, grid, arch or catenary, compatible with the actual deformation pattern, may be delineated, and the members and their joints designed for the resulting forces thereon. The skeleton (or truss/arch/catenary) may be either explicit and externally visible, as in a real truss, or implicit and embedded within a member, as in the case of the truss analogy for shear design of concrete beams and the truss analogy for plate girder design. Technology advancement forces Fibre Reinforcement Polymer (FRP) composites as vital building material especially under aggressive environmental conditions. FRP material is currently used as reinforced for concrete structures in which corrosion protection is a primary concern. FRP material is corrosion resistant and exhibit several properties that make them suitable as structural reinforcement.

Fibre Reinforced Polymer (FRP) materials are well recognized as a vital constituent of the modern concrete structures. The superiority of the FRP materials, in comparison with other conventional building materials like steel, cast iron and reinforced concrete, lies in its improved structural performance, in terms of stability, stiffness, strength (including improved resistance to fatigue loading) and durability [1-3]. Commercially, these FRP materials take the form of cables, sheets, plates etc. FRP materials are now being used in large numbers in many countries, including India, where, after a nascent beginning in 1950, the manufacture of FRP materials has become a major industry. In this context, the need for economical and reliable designs of FRP reinforced concrete structure gains importance. Most of the research efforts, particularly in the early years, have been directed towards the strengthening of masonry and concrete structures with FRP strips, sheets and fabrics [4, 5]. Significant research work (experimental as well as theoretical) has also been achieved in strengthening and retrofitting of diseased concrete structures [6, 7]. However, the use of Hybrid FRP reinforcements in concrete structures, especially the joint behaviour and formulation of design specifications are not well explored and remains still as research area.

From the literature review, it is understandable that enough work has been done in the area of GFRP wrappings of beam-column joints for retrofitting and rehabilitation purposes but the study on the use of non-metallic bars for beam-column joint applications is very scarce. Recently, non-metallic reinforcements are finding greater importance in structural applications in abroad. Despite their successful introduction into the construction industry, widespread acceptance of non-metallic reinforcements by the engineering industry depends on timely development of design guidelines and specifications.

This introduction chapter covers the classification of joints, failure mechanism, Strut and tie model, Glass fibres, manufacturing process and their structural application.

### 1.2. Classification of Joints

The joint is defined as the portion of the column within the depth of the deepest beam that frames in to the column. In a moment resisting frame, three types of joints can be identified (i) Interior joint (ii) Exterior joint (iii) Corner joint.

#### 1.2.1. Interior Joint

When four beams frame in to the vertical faces of a column, the joint is called as an interior joint. The effect of loads on the joints, the forces on an interior joint subjected to gravity loading can be depicted as shown in Figure 1a. The tension and compression from the beam ends and axial loads from the columns can be transmitted directly through the joint. In the case of lateral (or seismic) loading, the equilibrating forces from beams and columns as shown in Figure 1b develop diagonal tensile and compressive stresses with in the joint. Cracks develop perpendicular to the tension diagonal A-B Figure 1b in the joint. The compression struts and tension ties are shown by dashed lines and solid lines.

![Figure 1. Interior joint](image-url)
1.2.2. Corner Joint

In a beam when each frames in to two adjacent vertical faces of a column, then the joint is called as a corner joint. Wall type corners from another category of joints where in the applied moments tend to either close or open the corners. Such joints may also be referred as knee joints or L-joints. The stresses and cracks developed in such a joints are shown in Figures 2(a, b, c).

A typical knee joint, subjected to an “opening” bending moment and corner responding actions shown in Figure 3 a, b. It may be noted that the joint is usually subjected to axial forces and shearing forces, in addition to the bending moment. Opening corner joints tend to develop nascent cracks at the re-entrant corner and failure made by the formation of a diagonal tensile crack.

A typical knee joint, subjected to a “closing” bending moment and corner responding actions shown in Figure 4 a, b. The forces developed in a closing joint are exactly opposite to those in an opening corner joint. The major crack is oriented along the corner diagonal. These joints show better efficiency than the opening joints.

ACI–ASCE committee recommendations classify the beam-column joint in two categories based on loading conditions and anticipated deformation.

*Type 1 joints*: These are designed on the basis of strength without considering special ductility requirements.

*Type 2 joints*: These are designed to have sustained strength under deformation reversals in to inelastic range.

Any joint in structural frame designed to resist gravity and normal wind loads falls in to type-1 category. Joint in framed structures designed to resist lateral loads due to earthquake, blast and cyclonic winds falls in to type 2 categories.
1.3. FRP Materials

FRP materials are generally classified according to resin type, fibre type and fibre architecture. The various types of FRP reinforcements available in the market are shown in Figure 5.

![Figure 5. Various types of FRP reinforcements](image)

A report for the use of FRP composites as an internal reinforcements for flexural members. Based on this report, the following points have been arrived at (i) The mechanical properties of FRP bars are typically quite different from those of steel bars, (ii) FRP bars have lesser weight, lower young's modulus but higher strength than steel, (iii) The density of FRP bars is found to be one sixth to one fourth lesser than that of steel [8]. The bond mechanism of FRP reinforcements with concrete under cyclic loading conditions. FRP reinforcements in five different forms are surrounded in concrete blocks and are subjected to a stress level at service nearly 4, 50,000 cycles. Pull out tests have been conducted at the end of the cyclic loading [9]. A simplified reinforced constitutive model based on smeared cracks concept. The model is combined by a compression, a tension and a shear constitutive model for concrete and a constitutive model for steel bar. The simplified constitutive model has fewer parameters and simpler hysteretic loops [10].

The seismic performance of joint in lightly reinforced concrete frames were studied. Mainly focussed on effect of joint rotation, column axial load, cross-reinforcement in the joint and percentage of longitudinal reinforcement in the beam [11]. The beam-column joint model under reversed-cyclic loading which provides a simple representation of the primary inelastic mechanisms. The mode of failure mainly due to anchorage failure of beam [12]. A new analytical model for beam column joint to represent the shear behaviour within the joint panel zone by establishing shear stress - shear deformation history envelope with salient response points, which forms the backbone for the primary curve of the strength - deformation model for joints in non - linear dynamic analysis computational tool [13]. In RC Beam-column joints were cast with adequate and deficient bond of reinforcements. FRP sheets and strips were applied on the joints in different configurations and the columns are subjected to an axial force while the beams are subjected to a cyclic load with well-ordered displacement [14]. The behaviour of exterior beam-column joints turned out to be different from that of interior connections [15]. Various types of joints under seismic action on the critical parameters that affect joint performance mainly bond strength and transfer of shear [16]. Design procedures for the RC beam-column joints under seismic loading condition with a distinct prominence on three international codes of practice viz. ACI 318M-02, NZS 3101: 1995 and EN 1998-1: 2003 [17]. The mode of failure mainly due to concrete crushing or fiber-reinforced polymer (FRP) fracture or deboning [18]. Two types of repeated loading schemes such as constant amplitude fatigue loading (scheme I) and accelerated fatigue loading with variable amplitude (scheme II) are taken for investigation [19]. The use of the non-corrodirble fiber-reinforced polymer (FRP) reinforcing bars in parking garages and road overpasses in extreme weather conditions is beneficial to overcome the steel-corrosion problems [20].

The joint without transverse reinforcement exhibits brittle behaviour during an earthquake [21]. In the Bhuj earthquake in Gujarat, the main reason for the failure of most of the structures was the failure of the beam column joints [22]. The experimental studies are confirmed with the analytical studies carried out by finite element models using ANSYS. The ferrocement playing important role for strengthened beam-column joints, it gives better structural performance compared to un-strengthened specimens [23]. Based on the experimental evidences, it can significantly affect nonlinear behaviours, i.e. bond mechanism and shear failure of joints [24]. The reduced effect of axial loading level and longitudinal steel ratio in the ultimate load level when the specimens were strengthened with ferrocement [25]. The combine the demand of the conservative use of materials and the safety requirements. The structure are needed to be designed having sufficient durability and safety as well as consuming minimum amount material. These requirements of a structural design are contradictory to each other and should be optimized [26]. The RC structures with composite materials has been widely increased in the present scenario, because of its light weight, high impact value and flexibility [27]. The shear behaviour of beam -column joints with special captivity in the joint region along with different reinforcement detailing for anchorage of beam bars, confinement in joint and additional reinforcement in beam and column, further Glass Fiber Reinforced Polymer (GFRP) is used as an external confinement [28]. The experimental behavior of full-scale beam-column space (three-dimensional) joints under displacement-controlled cyclic loading. Beam column joint subjected to seismic force will experience large shear forces, diagonal tension and high bond stresses.
in the reinforcement bars [29]. Hence distinctive attention need to be given for design and construction of beam column joint subjected to seismic loading. Based on the location of joint, beam-column joint are classified as interior, exterior and corner joint [30].

1.4. Objectives and Scope

Objectives of this present study are given below:

- To create a data set to form the mechanical properties of concrete, steel, GFRP reinforcements and bond properties between concrete and steel/GFRP.
- Validate the data set with already available data from literature.
- To simulate the model using ABAQUS software with the help of the following elements; Continuum Solids and rebar elements.
- To conduct the experiments to find the behaviour of beam-column joints reinforced with conventional steel and GFRP reinforcements.
- To study the joint behaviour of Joints under monotonically increasing static and cyclic load conditions.
- Interpretation of analytical findings with results obtained from experiments.

Scope of this present study is restricted to:

- Analysis of Exterior Beam-Column joint reinforced with GFRP reinforcements.
- Analysis of Beam-Column joint under static and cyclic load conditions.
- Analysis is restricted to material non-linearity.

2. Experimental Investigations

2.1. Test Program

2.1.1. Specimens

The experimental investigation consists of twelve reinforced concrete beam-column joint with various parametric conditions. The test program consists of three series of test (Series A, B and C). The specimens and their properties, loading conditions and the test parameters are listed below.

2.1.2. Test Series A

Test Series A consists of four beam-column joint specimens with identical dimensions, geometry and reinforcing arrangement. The reinforcements used in the experiment are varied vide, Threaded GFRP, sand coated GFRP, grooved GFRP, and the conventional steel. The specimen consists of beam of size 150 mm × 200 mm and column of size 200 mm × 150 mm. The height of the column is 2000 mm and the length of the beam is 1000mm. Figure 6 shows the typical beam column joint with all reinforcement details.

Figure 6. Typical beam-column joint with reinforcement details
2.1.3. Test Series B

Test Series B consists of four beam-column joint specimens with identical dimensions, geometry and reinforcing arrangement with two different concrete grades. The first specimen is of grade M20 and the next specimen is of grade M30. The beam and the column are reinforced with conventional steel as well as GFRP reinforcements used in this study.

2.1.4. Test Series C

Test Series C consists of four beam-column joint specimens with identical dimensions, geometry based on the provision of the shear reinforcements to the joint region. The specimens are reinforced in such a way that the joints are having with and without stirrups. The reinforcement details for both type of reinforcements are shown in Figures 7 and 8.

![Beam-column joint with stirrups](image1)

(a) GFRP  (b) Steel

Figure 7. Beam-column joint with stirrups

![Beam-column joint without stirrups](image2)

(a) GFRP  (b) Steel

Figure 8. Beam-column Joint without stirrups

2.1.5. Specimen Details

- BCJSFT – Beam column joint with Threaded GFRP bars;
- BCJSFG – Beam column joint with Grooved GFRP bars;
- BCJSFS – Beam column joint with Sand coated GFRP bars;
- BCJS – Beam column joint with steel reinforcement;
- BCJS20 – Beam column joint with steel reinforcement with M20 grade concrete;
- BCJSFT20 – Beam column joint with Threaded GFRP bars with M20 grade concrete;
- BCJS30 – Beam column joint with steel reinforcement M30 grade concrete;
- BCJSFT30 – Beam column joint with Threaded GFRP bars with M30 grade concrete;
• BCJS1 – Beam-Column Joint with Steel reinforcement without joint stirrups;
• BCJFT1 – Beam-Column Joint with GFRP reinforcement without joint stirrups;
• BCJS3 – Beam-Column Joint with Steel reinforcement with joint stirrups;
• BCJFT3 – Beam-Column Joint with GFRP reinforcement with joint stirrups.

2.2. Construction

All the form works were made with steel panel. They are positioned in horizontal direction. The cages of reinforcements are installed in the formwork and mixed concrete was placed in the formwork. Figure 9 shows a typical beam column joint specimen casted in the mould with strain gauges installed in it.

![Figure 9. Typical beam – column Joint specimen ready for testing](image)

2.3. Reinforcements

Main reinforcements of column and beam were made from high yield strength deformed steel bars of 12 mm diameter for control specimens and GFRP bars of same diameter used for GFRP specimens. The control specimen is reinforced with 4 nos. of 12 mm dia. and 8mm dia. stirrups for beam and 4 nos. 12mm dia. and 8 mm dia. ties for column.

2.4. Mechanical Properties of Materials

2.4.1. Concrete

Concrete mix M20, M25 and M30 were designed as per IS recommendations. PPC was used for concrete. Locally available sand having fineness modulus of 2.75 and specific gravity of 2.57 was used as fine aggregate. Crushed stone with maximum size 20 mm having fineness modulus of 7.15 and specific gravity of 2.73 was used as coarse aggregate. Average compressive strength of 150 mm size cubes after 28 days of moist curing was found to be 38 N/mm$^2$, 44.22 N/mm$^2$ and 52 N/mm$^2$ respectively.

2.4.2. Steel and GFRP

The various properties of reinforcements that are related to the present study are investigated through laboratory experiments and the results are presented in table 1. The stress strain curve for HYSD bar and GFRP bars are shown in Figures 10 and 11.

<table>
<thead>
<tr>
<th>Properties</th>
<th>GFRP rod (Threaded)</th>
<th>GFRP rod (Sand coated)</th>
<th>GFRP rod (Grooved)</th>
<th>Steel Fe 415 rod (S)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Tensile strength (MPa)</td>
<td>600</td>
<td>690</td>
<td>525</td>
<td>448.2</td>
</tr>
<tr>
<td>Longitudinal modulus (GPa)</td>
<td>42.86</td>
<td>57.5</td>
<td>35</td>
<td>207.5</td>
</tr>
<tr>
<td>Strain</td>
<td>0.014</td>
<td>0.012</td>
<td>0.015</td>
<td>0.002</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.22</td>
<td>0.22</td>
<td>0.2</td>
<td>0.3</td>
</tr>
</tbody>
</table>
2.5. Experimental Setup and Instrumentation

The experimental setup and the instrumentation for a typical beam column joint are shown in Figures 12 and 13. Displacements on beam are measured by LVDT at the loading point and at the mid span. Another LVDT is used to measure the deflection at the mid height of the column. A hydraulic jack (capacity 10 tonnes) was used for applying a vertically downward force on top face of the beam at distance 90 mm from beam-column interface producing moment and shear force on the joint. The load increments applied on the beam was 1kN. The readings of the LVDT are recorded at regular intervals from the displacement indicator during the loading. Demec readings are also measured using Demec Gauge at five points across the joint interface. After widening of cracks, when deflections started increasing very fast, dial gauges are removed and load on the beam is increased till failure. The specimen is considered as failed when load applied on beam started decreasing. Observations are also made regarding crack formation as well as the mechanism of failure of the joint. The crack pattern for steel and GFRP specimens are shown in Figure 14 and 15.
2.6. Load-deflection Relationship

Based on the experiment results load-deflection relationship curve have been plotted and the following Figures 16 and 17 shows the load-deflection relationship for control specimen and GFRP specimen.

Figure 16. Experimental load-deflection relationship BCJS-1 and BCJS-3

Figure 17. Experimental load-deflection relationship BCJFT-1 and BCJFT-3
3. Finite Element Software (ABAQUS)

ABAQUS is a suite of powerful engineering simulation programs, based on the finite element method, which can solve problems ranging from relatively simple linear analyses to the most challenging nonlinear simulations. In a nonlinear analysis ABAQUS automatically chooses appropriate load increments and convergence tolerances.

3.1. Finite Element Modelling of Beam-Column Joint

To study the behaviour of the beam column joint, the specimens were modelled and analyzed using a Finite Element Software ABAQUS using the above said element types and the material properties. The models developed in ABAQUS software with conventional and non-conventional reinforcement detailing. An axial load of 10 kN is applied on the column with fixed base and a roller support at the top. The load on the beam is applied at a distance of 100 mm from the cantilever end. The models were analyzed for both the monotonic and cyclic loadings.

3.1.1. Sectional Properties

The parameters to be considered for Solid element (C3D8R) are material name, material dimensions and orientation angles (in X and Y direction). The parameters to be considered for Rebar element are cross sectional area and initial strain. The values were entered for rebar element as given in the table 2 and ABAQUS model shown in Figure 18.

<table>
<thead>
<tr>
<th>Element name</th>
<th>Element type</th>
<th>Particulars</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebar main</td>
<td>Rebar (Main bars of the Beam)</td>
<td>Cross Sectional Area</td>
<td>113.097 mm$^2$</td>
</tr>
<tr>
<td>Rebar column</td>
<td>Rebar (Main bars of the Column)</td>
<td>Cross Sectional Area</td>
<td>113.097 mm$^2$</td>
</tr>
<tr>
<td>Rebar beam str</td>
<td>Rebar (Beam Stirrups)</td>
<td>Cross Sectional Area</td>
<td>50.265 mm$^2$</td>
</tr>
<tr>
<td>Rebar col ties</td>
<td>Rebar (Column Lateral Ties)</td>
<td>Cross Sectional Area</td>
<td>50.265 mm$^2$</td>
</tr>
</tbody>
</table>

3.1.2. Material Properties

The material used in this problem is the concrete material and is assumed to be linearly elastic. Thus a single linear elastic material is created for the model. The properties that are to be defined for the solid continuum model are Mass density, Young’s modulus, Poisson’s ratio, concrete damaged plasticity, concrete smeared cracking and Mohr coulomb plasticity. For the reinforcing bars, the yield stress $f_y = 432$ MPa and the tangent modulus is 847 MPa. The concrete cube compressive strength $f_{ck}$ is 44.22 MPa, 80% of which is used as the cylinder strength (Table 3 and 4).
Table 3. Material properties

<table>
<thead>
<tr>
<th>Element Type</th>
<th>Young’s Modulus</th>
<th>Poisson’s Ratio</th>
<th>Yield Stress</th>
<th>Longitudinal Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebar for steel</td>
<td>2.1×10^{11} N/m²</td>
<td>0.3</td>
<td>448.2×10^6 N/m²</td>
<td>207×10^6 N/m²</td>
</tr>
<tr>
<td>Rebar for GFRP</td>
<td>6.0×10^{10} N/m²</td>
<td>0.2</td>
<td>1.3×10^8 N/m²</td>
<td>42.86×10^6 N/m²</td>
</tr>
<tr>
<td>Solid C3D8R Concrete</td>
<td>32517×10^6 N/m²</td>
<td>0.15</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Property</th>
<th>GFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2100</td>
</tr>
<tr>
<td>Longitudinal modulus (GPa)</td>
<td>39</td>
</tr>
<tr>
<td>In-plane shear modulus (GPa)</td>
<td>3.8</td>
</tr>
<tr>
<td>Longitudinal tensile strength (MPa)</td>
<td>1080</td>
</tr>
<tr>
<td>Transverse tensile strength (MPa)</td>
<td>39</td>
</tr>
<tr>
<td>In-plane shear strength (MPa)</td>
<td>89</td>
</tr>
<tr>
<td>Ultimate longitudinal tensile strain (%)</td>
<td>2.8</td>
</tr>
<tr>
<td>Ultimate transverse tensile strain (%)</td>
<td>0.5</td>
</tr>
<tr>
<td>Longitudinal compressive strength (MPa)</td>
<td>620</td>
</tr>
<tr>
<td>Transverse compressive strength (MPa)</td>
<td>128</td>
</tr>
</tbody>
</table>

It is required to consider the geometric non-linearity, when the material non-linearity is adopted. Hence, the large deformation effects have to be included in the analysis. Then the convergence difficulties will be eliminated to some extent. The convergence criteria for the displacement may be used instead of the convergence criteria for the force. It is known that, though the force convergence will give accurate results, the displacement convergence will be reliable where very accurate results need not be necessary.

3.1.3. Elements

C3D8R: The ELEMENT option deals with the element-nodal connectivity list. The element type is specified using the TYPE parameter. The choice of element type is as important as any other aspect of a finite element analysis. In this model the element used is C3D8R - 8-node linear brick element. In this the C represents Continuum and D represents Displacement H represents Hybrid (Figure 19).

![Figure 19. Continuum solid element](image-url)
**Rebar:** Rebar is used to define layers of uniaxial reinforcement in solid elements (such layers are treated as a smeared layer with a constant thickness equal to the area of each reinforcing bar divided by the reinforcing bar spacing) (Figure 20).

![Figure 20. Rebar model in 3D element](image)

**3.1.4. Meshing**

The finite element mesh is created for the model. ABAQUS uses a number of different meshing techniques. The default meshing technique assigned to the model. The Beam-Column Joint is modelled in different mesh sizes. It is suggested that finer mesh model will give accurate results and the coarser one will take less time for analysis (Figure 21).

![Figure 21. Meshing applied to the model](image)

**3.1.5. Post processing**

The post processing involves the visualization of the deformed model shape and also plotting the results (Figures 22 and 23).
The extensive modelling and implementation of finite element analysis for beam column joints. In order to assess the ability of the proposed finite element model to simulate the behaviour of beam column joints tested experimentally under monotonic loading conditions, a full scale beam column joints have been considered. A detailed material data required for modelling and analysis were carried out. The procedure for modelling are described with the different element, material properties. The comparative study between the experimental and the analytical results shows that the model created in the software is more reliable. Finally the results obtained from the FEM analysis are presented.

5. Theoretical Investigation

In the case of joints strut and tie concept is well suitable for the analysis to find the stress distribution at the joints. A new design equation based on strut and tie concept is proposed for predicting the joint shear strength of monotonically loaded exterior GFRP reinforced beam column joints. For this purpose, the influence of several key variables on the behaviour of beam-column joints are inspected using results of parametric studies on an experimental database compiled from a large number of exterior joint tests.

Present design guidelines:

The ACI-ASCE Committee and EC8 recommend the following design equations for the shear strength of monotonically loaded joints (Equations 1 and 2).

\[ V_{jd} = 1.058 \sqrt{f_c b_{eff} h_c} \quad (ACI – ACSE Committee 352) \]
\[ V_{jd} = 0.525 f_c^{2/3} b_{eff} h_c \] (EC8 Ductility class DCL) \hspace{1cm} (2)

In order to investigate the reliability of the above design equations, the authors carried out several parametric studies on monotonically loaded exterior beam-column joints.

5.1. Parametric studies on joint shear strength

The parametric investigation of exterior beam-column joint behaviour is carried out based on the previous tests available in the literatures, tests carried out in our laboratory and the present tests done in our laboratory. The loading in all the tests were monotonic. The monotonically loaded exterior beam column joints are investigated considering the test conditions. Examining a large number of individual series of tests as a single database has the advantage of observing which variables have a significant influence on joint shear strength in all tests and which variables interact with each other.

Table 5 shows the experimental database used in this study. The database comprises the results obtained in our laboratory and the other researchers like Ortiz [31], Kordina [32], Scott [33], Scott & Hamill [34], Taylor [35], and Parker & Bullman [36]. The specimen forms included in the database are shown in Figure 24 (a, b, c) shows the notations used in this study

<table>
<thead>
<tr>
<th>Investigator</th>
<th>Specimen</th>
<th>Detail</th>
<th>H mm</th>
<th>L mm</th>
<th>h_d/h_b</th>
<th>b_d/b_s</th>
<th>Beam rein. Ratio</th>
<th>Column rein. Ratio</th>
<th>f_c</th>
<th>Col axial load</th>
<th>Jpred/ Jactual</th>
<th>Failure modes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ortiz</td>
<td>BCJ 1</td>
<td>L bar</td>
<td>2000</td>
<td>1050</td>
<td>1.33</td>
<td>1</td>
<td>1.1</td>
<td>2.19</td>
<td>34</td>
<td>0</td>
<td>0.68</td>
<td>Joint shear -js</td>
</tr>
<tr>
<td></td>
<td>BCJ 2</td>
<td>L bar</td>
<td>2000</td>
<td>1100</td>
<td>1.33</td>
<td>1</td>
<td>1.1</td>
<td>2.19</td>
<td>38</td>
<td>0</td>
<td>0.77</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>BCJ 3</td>
<td>L bar</td>
<td>2000</td>
<td>1100</td>
<td>1.33</td>
<td>1</td>
<td>1.1</td>
<td>2.92</td>
<td>33</td>
<td>0</td>
<td>0.64</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>BCJ 4</td>
<td>L bar</td>
<td>2000</td>
<td>1100</td>
<td>1.33</td>
<td>1</td>
<td>1.1</td>
<td>3.65</td>
<td>34</td>
<td>0</td>
<td>0.78</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>BCJ 5</td>
<td>L bar</td>
<td>2000</td>
<td>1100</td>
<td>1.33</td>
<td>1</td>
<td>1.1</td>
<td>3.65</td>
<td>38</td>
<td>300</td>
<td>0.72</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>BCJ 6</td>
<td>L bar</td>
<td>2000</td>
<td>1100</td>
<td>1.33</td>
<td>1</td>
<td>1.1</td>
<td>3.65</td>
<td>35</td>
<td>300</td>
<td>0.68</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>BCJ 7</td>
<td>L bar</td>
<td>2000</td>
<td>1100</td>
<td>1.33</td>
<td>1</td>
<td>1.1</td>
<td>3.65</td>
<td>35</td>
<td>300</td>
<td>0.68</td>
<td>js</td>
</tr>
<tr>
<td>Kordina</td>
<td>RE 2</td>
<td>L bar</td>
<td>3000</td>
<td>1000</td>
<td>2</td>
<td>1</td>
<td>0.9</td>
<td>2.41</td>
<td>25</td>
<td>240</td>
<td>0.66</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>RE 3</td>
<td>L bar</td>
<td>3000</td>
<td>1000</td>
<td>1.5</td>
<td>1</td>
<td>1.8</td>
<td>2.41</td>
<td>40</td>
<td>400</td>
<td>0.96</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>RE 4</td>
<td>L bar</td>
<td>3000</td>
<td>1000</td>
<td>1.5</td>
<td>1</td>
<td>1.2</td>
<td>2.41</td>
<td>32</td>
<td>51</td>
<td>0.83</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>RE 6</td>
<td>L bar</td>
<td>3000</td>
<td>1000</td>
<td>1.5</td>
<td>1</td>
<td>1.2</td>
<td>2.41</td>
<td>32</td>
<td>213</td>
<td>0.91</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>RE 7</td>
<td>L bar</td>
<td>3000</td>
<td>975</td>
<td>1.4</td>
<td>1</td>
<td>1.3</td>
<td>1.61</td>
<td>26</td>
<td>650</td>
<td>0.87</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>RE 8</td>
<td>U bar</td>
<td>3000</td>
<td>975</td>
<td>1.4</td>
<td>1</td>
<td>1.3</td>
<td>1.61</td>
<td>28</td>
<td>525</td>
<td>0.9</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>RE 9</td>
<td>U bar</td>
<td>3000</td>
<td>975</td>
<td>1.4</td>
<td>1</td>
<td>1.3</td>
<td>1.61</td>
<td>28</td>
<td>770</td>
<td>0.86</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>RE 10</td>
<td>U bar</td>
<td>3000</td>
<td>975</td>
<td>1.56</td>
<td>1</td>
<td>1.2</td>
<td>1.61</td>
<td>24</td>
<td>551</td>
<td>0.94</td>
<td>js</td>
</tr>
<tr>
<td>Taylor</td>
<td>P1/41/24</td>
<td>L bar</td>
<td>1290</td>
<td>470</td>
<td>1.4</td>
<td>1.4</td>
<td>2.4</td>
<td>4.1</td>
<td>33</td>
<td>240</td>
<td>0.97</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>P2/41/24</td>
<td>L bar</td>
<td>1290</td>
<td>470</td>
<td>1.4</td>
<td>1.4</td>
<td>2.4</td>
<td>4.1</td>
<td>29</td>
<td>240</td>
<td>0.94</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>P2/41/24A</td>
<td>L bar</td>
<td>1290</td>
<td>470</td>
<td>1.4</td>
<td>1.4</td>
<td>2.4</td>
<td>4.1</td>
<td>47</td>
<td>240</td>
<td>0.92</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>A3/41/24</td>
<td>L bar</td>
<td>1290</td>
<td>470</td>
<td>1.4</td>
<td>1.4</td>
<td>2.4</td>
<td>4.1</td>
<td>27</td>
<td>240</td>
<td>0.88</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>D3/41/24</td>
<td>L bar</td>
<td>1290</td>
<td>470</td>
<td>1.4</td>
<td>1.4</td>
<td>2.4</td>
<td>4.1</td>
<td>53</td>
<td>60</td>
<td>0.89</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>B3/41/24</td>
<td>L bar</td>
<td>1290</td>
<td>470</td>
<td>1.4</td>
<td>1.4</td>
<td>2.4</td>
<td>4.1</td>
<td>22</td>
<td>240</td>
<td>0.92</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>C3/41/24BY</td>
<td>U bar</td>
<td>1290</td>
<td>470</td>
<td>1.4</td>
<td>1.4</td>
<td>2.4</td>
<td>4.1</td>
<td>32</td>
<td>240</td>
<td>1.04</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>C3/41/13Y</td>
<td>U bar</td>
<td>1290</td>
<td>470</td>
<td>1.4</td>
<td>1.4</td>
<td>4.1</td>
<td>28</td>
<td>240</td>
<td>0.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Scott</td>
<td>C1AL</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>1.1</td>
<td>4.29</td>
<td>33</td>
<td>50</td>
<td>0.87</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>41</td>
<td>275</td>
<td>0.89</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>C4A</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>44</td>
<td>275</td>
<td>0.86</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>C4AL</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>36</td>
<td>50</td>
<td>0.86</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>C7</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>2</td>
<td>1.36</td>
<td>1.4</td>
<td>4.29</td>
<td>35</td>
<td>275</td>
<td>0.9</td>
<td>js</td>
</tr>
<tr>
<td></td>
<td>C3L</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>35</td>
<td>50</td>
<td>1.03</td>
<td>js</td>
</tr>
<tr>
<td>Sample</td>
<td>Type</td>
<td>Age (days)</td>
<td>Water (%)</td>
<td>Steel (kg/m³)</td>
<td>Ultimate Strength (MPa)</td>
<td>Bending Moment (kN.m)</td>
<td>js</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>------</td>
<td>------------</td>
<td>----------</td>
<td>---------------</td>
<td>------------------------</td>
<td>-----------------------</td>
<td>----</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C6</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>40</td>
<td>275</td>
<td>1.05</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C6L</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>46</td>
<td>50</td>
<td>0.94</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C9</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>2</td>
<td>1.36</td>
<td>1.4</td>
<td>4.29</td>
<td>36</td>
<td>275</td>
<td>0.93</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C4ALN0</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>42</td>
<td>50</td>
<td>0.88</td>
<td>Punching</td>
<td></td>
</tr>
<tr>
<td>C4ALN1</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>46</td>
<td>50</td>
<td>0.85</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C4ALN3</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>42</td>
<td>50</td>
<td>0.78</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C4ALN5</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>50</td>
<td>50</td>
<td>0.85</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C4ALH0</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>104</td>
<td>100</td>
<td>0.86</td>
<td>p</td>
<td></td>
</tr>
<tr>
<td>C6LN0</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>51</td>
<td>50</td>
<td>0.92</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C6LN1</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>51</td>
<td>100</td>
<td>0.96</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C4ALH1</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>95.2</td>
<td>100</td>
<td>0.93</td>
<td>Bending</td>
<td></td>
</tr>
<tr>
<td>C4ALH3</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>105.6</td>
<td>100</td>
<td>0.97</td>
<td>b</td>
<td></td>
</tr>
<tr>
<td>C4ALH5</td>
<td>L bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>98.4</td>
<td>100</td>
<td>1</td>
<td>b</td>
<td></td>
</tr>
<tr>
<td>C6LN3</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>49</td>
<td>50</td>
<td>0.92</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C6LN5</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>37</td>
<td>50</td>
<td>0.74</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C6LH0</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.3</td>
<td>2.1</td>
<td>4.29</td>
<td>101</td>
<td>100</td>
<td>0.72</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C6LH1</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>102</td>
<td>100</td>
<td>0.98</td>
<td>js</td>
<td></td>
</tr>
<tr>
<td>C6LH3</td>
<td>U bar</td>
<td>1700</td>
<td>750</td>
<td>1.4</td>
<td>1.36</td>
<td>2.1</td>
<td>4.29</td>
<td>97</td>
<td>100</td>
<td>0.93</td>
<td>js</td>
<td></td>
</tr>
</tbody>
</table>

### Parker

<table>
<thead>
<tr>
<th>Sample</th>
<th>Type</th>
<th>Age (days)</th>
<th>Water (%)</th>
<th>Steel (kg/m³)</th>
<th>Ultimate Strength (MPa)</th>
<th>Bending Moment (kN.m)</th>
<th>js</th>
</tr>
</thead>
<tbody>
<tr>
<td>4a</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>0.9</td>
<td>1.09</td>
</tr>
<tr>
<td>4b</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>0.9</td>
<td>1.09</td>
</tr>
<tr>
<td>4c</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>0.9</td>
<td>1.09</td>
</tr>
<tr>
<td>4d</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>0.9</td>
<td>4.38</td>
</tr>
<tr>
<td>4e</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>0.9</td>
<td>4.38</td>
</tr>
<tr>
<td>4f</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>0.9</td>
<td>4.38</td>
</tr>
<tr>
<td>5a</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>0.9</td>
<td>2.67</td>
</tr>
<tr>
<td>5b</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>0.9</td>
<td>2.67</td>
</tr>
<tr>
<td>5d</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>1.4</td>
<td>2.67</td>
</tr>
<tr>
<td>5e</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>1.4</td>
<td>2.67</td>
</tr>
<tr>
<td>5f</td>
<td>L bar</td>
<td>2000</td>
<td>850</td>
<td>1.67</td>
<td>1.2</td>
<td>1.4</td>
<td>2.67</td>
</tr>
</tbody>
</table>

### Present study

<table>
<thead>
<tr>
<th>Sample</th>
<th>Type</th>
<th>Age (days)</th>
<th>Water (%)</th>
<th>Steel (kg/m³)</th>
<th>Ultimate Strength (MPa)</th>
<th>Bending Moment (kN.m)</th>
<th>js</th>
</tr>
</thead>
<tbody>
<tr>
<td>BCSI SF</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI SF G</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI SF S</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI S</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI S20</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI SFT20</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI S30</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI SFT30</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI S1</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI FT1</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI S3</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
<tr>
<td>BCSI FT3</td>
<td>L bar</td>
<td>2000</td>
<td>1000</td>
<td>1</td>
<td>1</td>
<td>2.26</td>
<td>2.26</td>
</tr>
</tbody>
</table>
The theoretical expressions for the exterior beam-column joint reinforced with GFRP reinforcements. Firstly, the equation proposed considers the influence of beam longitudinal reinforcement ratio, which was not taken into account in previously suggested design equations. Secondly, as the influence of this parameter is taken into account, a more realistic estimate of the influence of joint aspect ratio is obtained. Thirdly, the influence of stirrups is considered differently for joints with low, medium and high amount of stirrup ratios, in a way, which was not considered in previously suggested equations. Finally, the proposed design equation predicts the joint shear strength of exterior beam column connections accurately with minimal standard deviation and is more reliable than the previously suggested equations. Based on the detailed experimental and FEM analysis, a new equation is proposed at the end of the chapter for the determination of joint shear strength for GFRP reinforced beam-column joints.

Figure 24. (a) Typical specimen shape in the experimental database; (b) Typical elevation and notations used for exterior beam column joints; (c) The strut and truss mechanisms
6. Conclusions

Based on the experimental, theoretical and analytical works the following conclusions are drawn.

- Providing GFRP bars with L steel clamps changes the failure modes of monotonically loaded exterior beam-column joints from joint shear to beam failure. Furthermore, the joint shear strength of joints is increased by 15% if detailed by L bars bent down detail beam reinforcement.
- Anchorage failures are not anticipated in joints with and without stirrups in monotonically loaded exterior beam column joints, with the provision of anchorage coupler at the joint in the GFRP reinforced specimens.
- Column axial compressive load has no influence on ultimate shear capacity of the joint but higher column compressive axial load and high column longitudinal reinforcement ratios are necessary for the concrete joints to avoid column failures.
- Joint shear strength values obtained from the experiments are comparable with the analytical results for both the beam-column joints reinforced with steel and GFRP reinforcements. The joint shear strength predicted by the analytical tool ABAQUS is also validated with experimental results.
- Increasing the beam longitudinal reinforcement ratio increases the joint shear strength. Because the influence of beam longitudinal reinforcement ratio is taken into account in the proposed equation predicts that the joint shear strength.
- The analysis and design of beam-column joints reinforced with GFRP reinforcements is carried out by strut and tie model. Strut and Tie models are based on the models for the steel reinforced beam-column joints. The resulting strut and tie model developed for the GFRP reinforced beam-column joints predicts joint shear strength for varying beam longitudinal reinforcement ratio, joint aspect ratio and stirrup ratio.
- The GFRP reinforced specimens performance is low when compared to the conventional steel reinforcements. But the use of hybrid reinforcements will give similar results as that of steel because of the very high elastic modulus of the hybrid reinforcements.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


The Effect of Using Sustainable Materials on the Performance-Related Properties of Asphalt Concrete Mixture

Amjad H. Albayati a, Waleed Arrak Turkey b*

a Professor, Civil Engineering Department, Baghdad University, Baghdad, Iraq.
b M.Sc. Student, Civil Engineering Department, Baghdad University, Baghdad, Iraq.

Abstract

Sustainability is very important in this world at this time. One of the best materials used for sustainability in asphalt concrete pavements is the warm mix asphalt (WMA) as well as the reclaimed asphalt pavement (RAP). WMA technology has the ability to reduce production temperature to reduce the fuel usage and emissions. RAP is the old concrete asphalt mixture that is out of service and using it again leads to preservation of the virgin material. This search studied the viability of using WMA with different percentages of RAP (10%, 30%, and 50%) and compared them with control hot mix asphalt (HMA) and WMA. The Marshall properties, Tensile strength ratio (TSR), rut depth and fatigue life were determined in this work. The results showed that the tensile strength ratio (TSR) for HMA was better than that for WMA by 6%, rut depth for HMA was (4.37 mm) lower than that for WMA was (6.5 mm), better fatigue life was obtained for WMA was (700 cycle) as compared to HMA was (500 cycle). In case of WMA with RAP (WMA-RAP), when the percentage of RAP increased with WMA, the moisture damage resistance improved by 2.5%, 13.3% and 15.4% for G1, G3 and G5 respectively, also the rutting resistance improved by 34.6%, 48% and 62.3% for G1, G3 and G5 respectively, but deteriorated of fatigue life by 45.8%, 74% and 88.5% for G1, G3 and G5 respectively.

Keywords: WMA; RAP; Marshall's Properties; TSR, Wheel Truck; Rutting Resistance; Flexural Beam Fatigue Testing; Fatigue Resistance.

1. Introduction

The Warm Mix Asphalt (WMA) is a novel method to save energy and environment-friendly protection by lowering the mixing and compaction temperatures, with relatively lesser consumption of energy and emission of exhaust when compared with the conventional hot mix asphalt (HMA). The WMA technologies have the ability to decrease production temperature (15°C to 40°C) [1]. As shown in Figure 1, in recent years, environmental protection is becoming an important factor in transport engineering, especially asphalt production. Even with the HMA used significantly in all the countries, some current researches recommend using different technologies that decrease the production temperature of asphalt mixtures. This technology is called the WMA, and it is generally used in the United States [2]. RAP is a method of reuse of materials from the existing asphaltic roads that has no ability to serve the future traffic. RAP can be used a sustainable technology, because it reduces emissions and reduces economic cost. When pavement mixtures deteriorate, they can be removed or recycled. The utilization of RAP can be considered environmentally friendly, because this technology reduces the use of natural resources and may perform better than the virgin asphalt mixtures [3].

*Corresponding author: waleedturky727@gmail.com

http://dx.doi.org/10.28991/cej-2019-03091444

© 2019 by the authors. Licensee C.E.J, Tehran, Iran. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license (http://creativecommons.org/licenses/by/4.0/).
2. Background

The manufacture of HMA pavements has devolved from mixing by hand to good automatic equipment, which tracks and maintains the quality of materials. Throughout this period, the belief has been that pavement performance depends on temperature control. Temperature is important for a good coating of aggregate, compaction, and stability. To improve that, increase in production temperature is the most common method. Despite its benefits, increased production temperature increases emissions and energy costs [5]. Techniques developed to save energy, reduce temperature of production, and reduce emissions, have previously been used in asphalt manufacturing. The way to determine appropriate Compaction temperature includes a comparison of the volumetric Density or air void content in compacted WMA samples and control HMA [6].

In the United States, in 1956, in the University of Iowa Ladies Csanyi, foamed asphalt was used as a soil binder. In 1968, cold water was added instead of steam by Mobil Oil Australia. This process made the foaming processes better economically [7]. Jenkins et al. (1999) discovered a new method that included half-warm foamed asphalt treatment. Jenkins investigated the concept and benefit of preheating the aggregate to a temperature above ambient level and below 100 °C, before adding foamed asphalt. The results exhibited a good coating of particles, cohesion of the mix, tensile strength, and compaction [8]. López et al. (2017) studied the mixing and compaction process of WMA, manufactured with super-stabilized, emulsified and assessed their characteristics in relation to the HMA [9]. Raab et al. (2017) stated the aging behavior and performance of several WMA-cutback asphalt roadways causes a decrease in the rut depth with ageing [10]. Albayati (2018) used zeolite to manufacture WMA in plant and evaluation this process [11]. Sarsam (2018) studied the behavior of WMA mixture under moisture damage [12]. Mahdi et al. (2019) studied Moisture Damage of Warm Mix Asphalt Concrete [13].

3. Research Methodology

The program of this work can be seen in by a flow chart as shown in Figure 2.

![Figure 1. Types of asphalt mixtures by temperature [4]](image)

![Figure 2. Flow Chart for Research Methodology](image)
4. Materials

The materials utilized in this article consist of asphalt cement as well as aggregate, mineral filler to produce HMA, for the production of WMA zeolite additives were used besides the materials mentioned above, and a reclaimed asphalt mixture were used for the production of both types of asphalt concrete mixture as a partial replacement for virgin materials.

4.1. Asphalt cement

In this study, the asphalt cement (40–50) was used. It was brought from the Doura refinery. All test results met the Iraqi specification [14]. Physical properties of this type are shown in Table 1.

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Designation No.</th>
<th>Value</th>
<th>SCRB Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration, 1/10 mm, 25°C, 100 g, 5 sec</td>
<td>D5</td>
<td>44</td>
<td>40–50</td>
</tr>
<tr>
<td>Softening Point, (c ring &amp; ball)</td>
<td>D36</td>
<td>48</td>
<td>-</td>
</tr>
<tr>
<td>Ductility, cm (25°C, 5 cm/min)</td>
<td>D113</td>
<td>125</td>
<td>&gt; 100</td>
</tr>
<tr>
<td>Specific Gravity, 25°C</td>
<td>D70</td>
<td>1.04</td>
<td>-</td>
</tr>
<tr>
<td>Flash Point, C, Cleveland open cup</td>
<td>D92</td>
<td>269</td>
<td>&gt; 232</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>After thin film oven test proprieties D1754</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retained Penetration of Residue,% (25°C, 100 g, 5 sec)</td>
</tr>
<tr>
<td>Ductility of Residue, cm (25°C, 5 cm/min)</td>
</tr>
<tr>
<td>Loss on Weight% (163°C, 50 g, 5 h)</td>
</tr>
</tbody>
</table>

4.2. Mineral Filler

A filler is a material that passes through sieve No. 200. The source of this material is the lime plant in the city of Karbala. The physical properties of the lime stone filler are shown in Table 2.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Result</th>
<th>SCRB Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.73</td>
<td>-</td>
</tr>
<tr>
<td>Passing Sieve No. 200 (0.075 mm),%</td>
<td>94</td>
<td>70–100</td>
</tr>
</tbody>
</table>

4.3. Aggregates

The aggregate used in this study was derived from the Al-Neibaie quarry, north of Baghdad. This type is generally utilized in Iraq for asphalt pavements. In this study the coarse aggregate is isolated from the fine aggregate followed by recombining it to the best possible extent with a filler to satisfy the Iraqi standard specification [14]. For the wearing layer. Routine tests were completed on the coarse, fine, and filler to assess their physical properties. Physical properties of the aggregate shown in Table 3 and the aggregate gradation and specification limits shown in Figure 3.

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Designation No.</th>
<th>Coarse Aggregate</th>
<th>Fine Aggregate</th>
<th>SCRB R/9 2003</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Specific gravity</td>
<td>ASTM C127 and C128</td>
<td>2.647</td>
<td>2.635</td>
<td>-----</td>
</tr>
<tr>
<td>Apparent Specific gravity</td>
<td>ASTM C127 and C128</td>
<td>2.666</td>
<td>2.655</td>
<td>-----</td>
</tr>
<tr>
<td>Percent water absorption</td>
<td>ASTM C127 and C128</td>
<td>0.13</td>
<td>0.524</td>
<td>-----</td>
</tr>
<tr>
<td>Percent wear (Los-Angeles Abrasion)</td>
<td>ASTM C131</td>
<td>19.7</td>
<td>-----</td>
<td>30 Max</td>
</tr>
<tr>
<td>Fractured pieces,%</td>
<td>-----</td>
<td>97</td>
<td>-----</td>
<td>90 Min</td>
</tr>
<tr>
<td>Sand Equivalent</td>
<td>ASTM D 2419</td>
<td>-----</td>
<td>56</td>
<td>45 Min. Super pave (SP-2)</td>
</tr>
<tr>
<td>Soundness loss by sodium sulfate solution,%</td>
<td>(C-88)</td>
<td>-----</td>
<td>3.6</td>
<td>12 Max</td>
</tr>
</tbody>
</table>
Zeolites are crystalline hydrated aluminium silicates. The material of zeolites is shown in Figure 4. Artificial Zeolite has been used in this study. This material has been tested, and an X-ray Fluorescence (XRF) device has been used to find its chemical composition, Figure 5 shows the (XRF) device that used to find the chemical composition of zeolite. The chemical composition of zeolite is given in Table 4.

<table>
<thead>
<tr>
<th>Chemical composition</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>32.2%</td>
</tr>
<tr>
<td>Al2O3</td>
<td>28.5%</td>
</tr>
<tr>
<td>Na2O</td>
<td>15.3%</td>
</tr>
<tr>
<td>L.O.I</td>
<td>24.0%</td>
</tr>
</tbody>
</table>

Reclaimed Asphalt Mixture

Reclaimed Asphalt Mixture was obtained from the road (Baghdad – Anbar) near the city of Fallujah, west of Baghdad. According to the directorate of roads and bridges of Anbar, this road was established in 1994 by a local company. The RAP was subjected to an extraction test to obtain the asphalt content and aggregate gradation. The result of the asphalt content and the physical properties of the aggregate are presented in Table 5. Also the aggregate gradation is shown in Figure 6.

Figure 3. Aggregate Gradation and Specification Limits

Figure 4. The material of zeolite

Figure 5. (XRF) device

Table 4. The Chemical Composition of Zeolite

Table 5. The Physical Properties of Aggregate

Figure 6. Selected Gradation and Specification Limits
Table 5. Properties of Aged Materials after Extraction Test

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt binder</td>
<td>Asphalt content</td>
<td>4.2%</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>Bulk specific gravity</td>
<td>2.552</td>
</tr>
<tr>
<td></td>
<td>Apparent specific gravity</td>
<td>2.590</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>Bulk specific gravity</td>
<td>2.585</td>
</tr>
<tr>
<td></td>
<td>Apparent specific gravity</td>
<td>2.819</td>
</tr>
<tr>
<td>Mineral filler</td>
<td>Percent passing sieve No. 200</td>
<td>99%</td>
</tr>
<tr>
<td></td>
<td>Specific gravity</td>
<td>2.81</td>
</tr>
</tbody>
</table>

Figure 6. Aggregate Gradation and Specification Limits for RAP mixture

5. Mix Design

To find the optimum asphalt content according to the Marshall method, five Marshall specimens were prepared with different asphalt contents (3.5%, 4%, 4.5%, 5%, and 5.5%, with an aggregate of the surface layer, and a Marshall test was used for this purpose. The average of the following values was taken to adopt the optimum asphalt content for wearing courses;

- Maximum bulk density
- Maximum stability
- Air voids at 4%

Hence, the optimum asphalt cement content was 4.6%.

6. Preparation of HMA and WMA

The fine and coarse aggregates were washed and dried to a constant weight at 110 °C and then cooled and separated according to the desired gradation. A mineral filler, material that passes through sieve No. 200 was used. To prepare the HMA, coarse and fine aggregate with the filler was recombined to meet the required gradation, according to the Iraqi Standard Specification [14]. Next, the total mixture of aggregate was heated to 160 °C before mixing it with asphalt cement. Asphalt cement was heated to a temperature that produced a viscosity of 170 ± 20 centistokes, after which the asphalt was added to the heated aggregate and mixed well until all the aggregate particles were coated with the asphalt cement. Preparation of WMA was different from HMA by compaction temperature, which was 115 °C. Also, in the case of the preparation of WMA (the zeolite was added to the heated aggregate) the mixture was mixed well for half a minute and then the asphalt was poured and the zeolite of 0.3% was added manually (by weight of the total mix) to the mixer. The aspha-min as well as the addition process is shown in Figure 7.
7. WMA-RAP Aggregate Gradation

Different percentages of RAP were mixed with WMA. These percentages were 10% of RAP with 90% of WMA, 30% of RAP with 70% of WMA, and 50% of RAP with 50% of WMA; the gradations of mixtures after mixing are shown in Figures 8 to 10. WMA were also prepared with the same gradations shown in Figures 8 to 10, where gradations, as shown in Figures 8 to 10, were equal to G1, G3, and G5, respectively. The performance of these mixtures was evaluated in terms of Marshall properties of moisture resistance, rutting resistance, and fatigue resistance.

Figure 8. Aggregate Gradation and Specification Limits for \((W + 10R) = G1\)

Figure 9. Aggregate Gradation and Specification Limits for \((W + 30R) = G3\)
8. Performance Testing and Results

8.1. Marshall Properties

The Marshall specimens were prepared, with 100 mm (4-in diameter) and 63 mm (2.5-in height) specimens, three replicates for each type of mixture. These were compacted at a temperature of 135°C in case of HMA and 115°C in case of WMA and WMA-RAP. The compaction was achieved by 75 blows per each face. Next, the compacted specimens were immersed in water as shown in Figure 11 at 60°C, for 45 minutes, and then the stability and flow were tested.

*Figure 11. Specimens in the Water Bath*

The Marshall stability in HMA was (8.5 KN) better than the Marshall stability in WMA was (6.7 KN) by 27% at the same gradation and same optimum asphalt content. As shown in Figure 12 the stability was increased by 16% in case of WMA after addition of 30% of RAP and by 17% after addition of 50% of RAP. Based on the Marshall flow values, the Marshall flow (standard (2 mm – 4 mm)) in HMA was (2.77 mm) less than in WMA was (2.85 mm) at the same gradation. Also as shown in Figure 12, when the RAP was added to WMA, a decrease in the value of the Marshall flow was observed. The mixture (W 50R) showed the lowest flow value when compared to other mixtures and when compared to WMA at the same gradation. This was on account of the mix becoming more stiff and brittle when the percentage of RAP was increased in it and also because the percentage of asphalt decreased after adding RAP. The results showed that the bulk density in HMA was (2.327 g/cm³) slightly better than that in WMA was (2.299 g/cm³). Also, as shown in Figure 12, the bulk density increased in the WMA by 8%, at the same gradation, after addition of 10% of RAP, by 39% after addition of 30% of RAP, and by 53% after addition of 50% of RAP.

*Figure 12. Stability and Flow Values of Marshall Specimens*

---

**Figure 10. Aggregate Gradation and Specification Limits for (W + 50R) = G5**
Moisture Susceptibility

The adopted procedure to evaluate the moisture susceptibility of WMA, WMA-RAP, and HMA specimens is AASHTO T 283-07. There were six specimens used for this test (three for the un-conditioned and three for the conditioned). They were prepared for all variables to an air void level ranging from 6% to 8%. The unconditional specimen was put in a water bath at 25°C, for 120 ± 10 minutes, to test it directly for indirect tensile strength (ITS). Conditional specimens were placed in frozen temperatures, at -18 ± 2°C, for 16 hours, and then in 60 ± 1°C for 24 hours. Three conditioned and three unconditioned specimens were tested using the Versa Tester Machine. The average value was computed as SII "ITS for moisture-conditioned specimens", and as SI "ITS for unconditioned specimens". The tensile strength ratio could be calculated from the Equation 1:

\[ TSR = \left( \frac{SII}{SI} \right) \times 100 \] (1)

Based on the results the WMA had a lower tensile strength ratio (TSR) was 77% than HMA was 82%. From results presented in Figure 13, it can be seen that After the addition of a percentage of RAP to WMA, the TSR of the WMA-RAP mixture considerably increased, becoming higher than the TSR of WMA, at the same gradation; by 3% after adding 10% of RAP, by 13% after adding 30% of RAP, and by 15% after adding 50% of RAP. This indicated that the material in the RAP was stiff and had higher strength. These results showed that RAP might increase the moisture resistance. This could be explained scientifically by the fact that the bond between the binder, which is a mix of virgin binder and aged binder, and the recycled aggregate was stronger than the bond between the virgin asphalt and virgin aggregate.
8.3. Rutting Resistance

The wheel-tracking machine was used to evaluate the rut depth according to EN 12697-22:2003 for WMA, WMA-RAP, and HMA. Two slabs were tested for each variable; the dimensions of the slab were (400×300×50) mm. The slabs were compacted by using Dyna-Comp Pneumatic Roller Compactor according to European Standard (EN 12697 – 33). Test temperature was 40°C for all type of mixes.

From the results, the rut depth at 10,000 cycles for HMA was (4.37 mm) lower than that of WMA was (6.5mm) by 33%. From the results in Figure14, after the addition of a percentage of RAP to WMA, the rut depth of the WMA-RAP mixtures decreased in comparison to WMA at the same gradation; by 35% after adding 10% of RAP, by 48% after adding 30% of RAP, and by 62% after adding 50% of RAP. This indicated the binder in RAP was more stiff and brittle because of aging. This was the reason for improvement in rutting resistance for WMA after addition RAP.

8.4. Flexural Beam Fatigue Testing

The Flexural Beam Fatigue Test estimates the cracking potential of the asphalt pavement due to repeated heavy traffic loading. The pneumatic repeated load system is used in this work for conducting the fatigue life test. This test includes one stress level of 20 psi, one temperature 20 ± 1°C, and Test Frequency, the time of loading (0.1 sec), and a rest time of 0.9 second, asphalt type. Slab specimens for flexural fatigue testing were compacted by using a roller compactor device and then the slab was cut to obtain beam specimens. The total number of beam specimens obtained from the sawed Water-Cutter Roller compacted slabs were used for the Flexural Fatigue test. The dimensions of the beams were a length of 38 cm, width of 5 cm, and a height of 6 cm. They were used in the constant stress level test. Figure 15 shows the Water-Cutter machine and Figure 16 shows the Fatigue Beam Specimens.
From results, it can be seen that the WMA mixture had better performance of fatigue resistance than the HMA by 29%, where the fatigue life was 500 cycle for HMA, and 700 cycle for WMA. As shown in Figure 17, after the addition of percentage of RAP to WMA, the fatigue life of the WMA-RAP mixtures decreased in comparison to the WMA, at the same gradation; by 44% after adding 10% of RAP, by 74% after adding 30% of RAP, and by 89% after adding 50% of RAP. This indicated the zeolite made WMA softer and the RAP made mixture made it more stiff and brittle, this was explained by [15]. Also because the percentage of asphalt in WMA-RAP mixtures, however the percentage of asphalt decrease in case of WMA-RAP because the percentage of asphalt in recycled material lowest from virgin material. It can be seen that fatigue life decreased as asphalt content decreased, because the decreasing thickness of the asphalt film led to an increase in tensile strain at the bottom of the layer, accordingly the fatigue life increased. This agreed with the findings of [16].

9. Conclusions

Based on limitation and the testing results, the following points could be concluded:

- HMA showed better moisture resistance than WMA by 6%. WMA-RAP mixtures showed improved moisture resistance, better than WMA, the addition of RAP to WMA showed much better moisture resistance than WMA. With zero percentage of RAP, the result showed TSR values for WMA with 10% RAP was (79%) higher than WMA was (77%), at the same gradation by 2.5%, for WMA with 30% RAP was (90%) higher than WMA was...
(78%), at same gradation by 13.3%, and for WMA with 50% RAP was (97%) higher than WMA was (82%), at same gradation, by 15.4%.

- Rut depth for WMA was 6.5 mm, for HMA was 4.37 mm at 10,000 cycles, and the rut depth of WMA was higher than HMA by 33%. After adding RAP, WMA improved the rutting resistance. The rut depth for G1 was 6.44 mm for WMA and 4.21 mm for WMA-RAP, which meant, the rut depth was reduced by 35% after 10% RAP was added. Rut depth for G3 was 6.2 mm for WMA and 3.22 mm for WMA-RAP, which meant, the rut depth was reduced by 48% when 30% of RAP was added. Rut depth for G5 was 4.49 mm for WMA and 1.69 mm for WMA-RAP, which meant, the rut depth was reduced by 62% when 50% RAP was added.

- The number of cycles to reach fatigue failure in WMA was higher than HMA by 29%. WMA-RAP had a lower number of cycles to reach fatigue failure than WMA; by 44% when 10% RAP was added, by 73% when 30% RAP was added, and by 88% when 50% RAP was added, and due to the mixture which contained the RAP became more brittle.

10. Conflicts of Interest

The authors declare no conflict of interest.

11. References


Sensitivity of Direct Runoff to Curve Number Using the SCS-CN Method

AG Soomro a, b, MM Babar a, A Memon a, b, A. Z. Zaidi a, A Ashraf b, J. Lund c
a US Pakistan Center for Advanced Studies in Water, Mehran University of Engineering and Technology, Jamshoro, Pakistan.
b Pakistan Agricultural Research Council.
c Department of Geography, The University of Utah, Utah, United States.

Abstract

This study explores the impact of runoff curve number (CN) on the hydrological model outputs for the Morai watershed, Sindh-Pakistan, using the Soil Conservation Service Curve Number (SCS-CN) method. The SCS-CN method is an empirical technique used to estimate rainfall-runoff volume from precipitation in small watersheds, and CN is an empirically derived parameter used to calculate direct runoff from a rainfall event. CN depends on soil type, its condition, and the land use and land cover (LULC) of an area. Precise knowledge of these factors was not available for the study area, and therefore, a range of values was selected to analyze the sensitivity of the model to the changing CN values. Sensitivity analysis involves a methodological manipulation of model parameters to understand their impacts on model outputs. A range of CN values from 40-90 was selected to determine their effects on model results at the sub-catchment level during the historic flood year of 2010. The model simulated 362 cumecs of peak discharge for CN=90; however, for CN=40, the discharge reduced substantially to 78 cumecs (a 78.46% reduction). Event-based comparison of water volumes for different groups of CN values—90-75, 80-75, 75-70, and 90-40—showed reductions in water availability of 8.88%, 3.39%, 3.82%, and 41.81%, respectively. Although it is known that the higher the CN, the greater the discharge from direct runoff and the less initial losses, the sensitivity analysis quantifies that impact and determines the amount of associated discharges with changing CN values. The results of the case study suggest that CN is one of the most influential parameters in the simulation of direct runoff. Knowledge of accurate runoff is important in both wet (flood management) and dry periods (water availability). A wide range in the resulting water discharges highlights the importance of precise CN selection. Sensitivity analysis is an essential facet of establishing hydrological models in limited data watersheds. The range of CNs demonstrates an enormous quantitative consequence on direct runoff, the exactness of which is necessary for effective water resource planning and management. The method itself is not novel, but the way it is proposed here can justify investments in determining the accurate CN before initiating mega projects involving rainfall-runoff simulations. Even a small error in CN value may lead to serious consequences. In the current study, the sensitivity analysis challenges the strength of the results of a model in the presence of ambiguity regarding CN value.

Keywords: SCS-CN Method; Initial Abstraction; Direct Runoff; HEC-HMS; Hydrological Model; Sensitivity Analysis.

1. Introduction

A thorough assessment of the hydrologic response of a watershed is essential for water resource planning and management at the watershed level. Models are the abstraction of real phenomena based on many assumptions. These assumptions are acceptable as far as they do not produce erroneous results. The accuracy of the assessment depends on
the proper choice of methods and procedures, the correctness of the input parameters, and the feasibility of the assumptions used. An accurate assessment will yield reliable discharges that may decrease risks of water-related hazards such as flood damages and water shortages.

In the mid-1930s, the US Soil Conservation Service (SCS) (now known as the Natural Resources Conservation Service) perceived an acute need for hydrologic data for the design of conservation practices. Owing to the intricacy of hydrologic systems and the inaccessibility of pertinent watershed data, the SCS developed the SCS-CN method to estimate rainfall-runoff or immediate water availability. The SCS-CN method is the function of the amount of precipitation, basin slope, soil type/condition, and land use and land cover [1]. It is a simple method intended to be easily accessible and adaptable for ungauged catchments.

The SCS conducted initial experiments, hiring three scientists: W.W. Homer, R.E. Horton, and R.K. Sherman. Horton (1933) worked on the model, categorizing infiltration capacity curves for initial abstraction and excess runoff [2], and Homer (1940) concentrated on the infiltration capacity (initial abstraction) of small watershed data [3]. In early 1940, the rainfall-runoff method was developed by Horner using field-collected infiltration data to simulate runoff volume. This method provided the basis for the SCS-CN technique, established in 1954. The SCS-CN method is the outcome of precise field observations from the late 1930s and early 1940s, followed by numerous scientific experiments. The work of Mockus, Sherman, Andrews, and Ogrosky recognized in the National Engineering Handbook (NEH-4) issued by the SCS in 1956, U.S. Department of Agriculture (USDA) [4-7]. Model improvements and revisions made in 1964, 1965, 1971, 1972, 1985, and 1993. The methods specified by Mockus and Andrews [4, 6] were interpreted by Cowan [8] to produce the present SCS-CN technique based on the water balance equation.

The Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS) includes the SCS-CN loss method to study the effects of a rainfall event in a watershed. In the HEC-HMS model, interception, evaporation, and infiltration processes are loss elements, and they are calculated using transform components [9]. The HEC-HMS model has been used successfully to estimate rainfall-runoff in small watersheds throughout the world (Razi et al. [10]. Knebl et al. (2005) assessed several hydrological models to predict flooding on a regional level and found the HEC-HMS model capable of simulating rainfall-runoff for different features of the watershed area [11]. Yener et al. (2006) successfully employed the HEC-HMS in the Yuvacik Basin of Turkey using occasion-based hourly data and intensity-duration-frequency (IDF) curves to derive rainfall-runoff scenarios [12], Shieh et al. (2007) used the HEC-HMS model in Taiwan to estimate the impact of check-dams on river discharge with satisfactory results [13]. Zorkeflee et al. (2009) researched the Sungai Kurau Basin for catchment management using GIS and the HEC-HMS model, revealing that the hydrologic behavior varies according to land use [14]. Verma et al. (2010) examined HEC-HMS and Water Erosion Prediction Project (WEPP) models to estimate rainfall-runoff in the Baitarani catchment of India and defended the suitability of the HEC-HMS model for this purpose [15]. Dastoran et al. (2011) determined the ability of HEC-HMS to simulate rainfall-runoff and observed that CN and initial loss are the most significant parameters affecting the results [16].

The above review was undertaken regionally to predict flood waves in the absence of precise CN values. This study employs the SCS-CN method in the HEC-HMS using measured precipitation data and evaluates the SCS-CN estimates for changing CNs through sensitivity analysis. The sensitivity analysis improves the confidence level and evaluates the effectiveness of a modeling approach and is also a major cause of success [17]. Hawkins et al. [18] contribute a hydrological database that proves that the Sensitivity (λ) and SCS-CN are used frequently in model designs. It has become known that the existing approaches for SCS-CN are overdesigned [19]. Tassew et al. [17] simulated flows using the hydrologic modeling approach of the HEC-HMS in Lake, Ethiopia. For the study purpose, they selected a catchment of 1609 km², ran the model, and projected runoff by accounting the losses and flow routing, employing SCS-CN. Initially, their results showed variability between the observed and the simulated peak flows. Finally, their sensitivity analysis showed that the CN played a sensitive role in the simulation of flows [18].

Many variables are used in rainfall-runoff estimations, and using hydrologic models to speculate the frequency of flood incidents adequately is always a challenging task where there is changing land use, land cover, and climate scenarios [12,19,20,21]. Selecting an appropriate CN is necessary for producing consistent results. CN values based on the watershed characteristics—land use (treatment), land cover (vegetation), soil (type and antecedent moisture situations) [22]. Experimental results show that many important decisions and expectations are made based on hydrologic modeling results, which are employed differently for each location. Appropriate parametric selection is thus essential for sustainable and realistic water resource management. Sensitivity analysis is an integral part of model development and involves an analytical examination of input parameters to aid in model validation and to guide future research.

In this study, we perform a sensitivity analysis to evaluate the impact of CN on direct runoff. We use the HEC-HMS hydrological model to develop a relationship between rainfall-runoff and CN for future water resource planning and development in the study area. The novelty of the study lies in the proposed methodology—the use of sensitivity analysis in regions with limited data and budgets to assess the necessity of additional investments for determining accurate CN values. If model results are not sensitive to CN, then there is no need to determine precise CN values. However, when
results are highly sensitive, then even a small error in CN value may have severe consequences.

2. Material and Methods

2.1. Study Area

This study designed for the Morai ungauged watershed, which spreads over 492.6 km² in the Kohistan, Khirthar mountain range of Sindh province, Pakistan (Figure 1). The watershed is located in an arid to semi-arid climate. The study area has numerous basins and sub-basins that rely on precipitation for water availability, as no irrigation infrastructure is in place due to its high altitude. The Kohistan region has experienced several devastating floods and droughts in the past 81 years (1933-2013). Droughts and flash floods have been prevalent in recent decades. The latest drought (1996-2002) harshly struck the study area, reducing water resources, agricultural productivity, groundwater levels, and water quality and increasing livestock deaths, mortality rates, migration rates, and poverty [23]. A historic flood took place in 2010, which resulted in human lives lost, destruction of properties and infrastructure, elimination of agricultural assets, breached field embankments, and damaged dug wells.

2.2. Data

The Water and Power Development Authority (WAPDA) of Pakistan provided precipitation data for the region from 1933-2013 from which the data for the flood year 2010 were used to achieve the study goals. The Advanced Land Observing Satellite’s (ALOS) Digital Elevation Model (DEM) at 30m resolution acquired through the US Geological Survey website. The United States-Pakistan Center for Advanced Studies (USPCAS-W) of Mehran University of Engineering and Technology, Jamshoro facilitated the use of Geographical Information System software (ArcGIS version 10.3.1).

2.3. Hydrologic Software

We conducted the rainfall-runoff modeling for the Thana Boula Khan (TBK) catchment using the HEC-HMS model and its GIS extension, HEC-GeoHMS. We brought the ALOS DEM into the ArcGIS interface (HEC-GeoHMS) to integrate spatial and hydrological parameters and to set up the basin’s physiographic database for the hydrologic model [24]. We employed the HEC-HMS model to simulate the initial abstraction and direct runoff, and we did not count the base-flow due to the selection of a precipitation-based flood event in the watershed. We used the Loss Model to estimate the runoff volume by calculating the losses due to initial abstraction, percolation, storage, and evaporation, deducting them from the precipitation. We used the Transform Model to reproduce the direct runoff through additional rainfall over the watershed, which required only lag time. We used the Muskingum Routing Model to model flood wave through the channel and produce flood hydrographs [25]. McCarthy [26] established a Muskingum method in 1938, which is the lumped flow routing technique we used in the current sensitivity analysis.

We used the default SCS-CN in HEC-HMS to simulate water availability through rainfall-runoff processes in the study area. In the HEC-HMS model, each parameter describes specific hydrologic conditions. We selected the CN that characterizes infiltration and interception for the sensitivity analysis. We detail the methodology in the flow chart (Figure 2).
2.4. Sensitivity Analysis

The sensitivity analyses based on three coefficients: absolute sensitivity, relative sensitivity, and sensitivity deviation. These coefficients offer a comparative investigation on the regional or the global scales [27]. Absolute sensitivity varies according to varying inputs and cannot be used to compare parametric sensitivity. The relative sensitivity is dimensionless and is useful for comparative parametric studies. Sensitivity deviation involves complex differentiation and is not widely used for hydrological model sensitivity analysis (Cunderlik and Simonovic, 2004) [28].

Because this study does not compare model sensitivity to various parameters, we use the absolute sensitivity troika coefficient (Equation 1) to assess the impact of the CN on excess runoff from precipitation events in the Morai watershed. For sensitivity analysis, the model developed for the study area was run multiple times for CN values from 40-90 (with increments of 5) to assess the influence of CN on direct runoff.

$$SA = \frac{\partial O}{\partial P}$$

Where; SA is the sensitivity analysis; ‘O’ is the output and ‘P’ is the input.

2.5. The Initial Loss Parameter (Li)

The initial and constant-rate technique involves three parameters: the initial physiography of the basin, existing soil properties, and the existing condition of basin land use (Equation 2).

$$R_{et} = \begin{cases} 0 & \text{if } \sum R_i < L_i \\ R_t - L_t & \text{if } \sum R_i > L_i \text{ and } R_t > L_i \\ 0 & \text{if } \sum R_i > l_i > \text{ and } R_t < L_i \end{cases}$$

Figure 2. Flow chart for sensitivity analysis
Where ‘\(R_e\)’ is the excess rainfall, ‘\(R_t\)’ is the rainfall depth during the time intermission Δt, ‘\(Lr\)’ indicating constant rate in mm all over the event, ‘\(R_i\)’ is the initial rainfall. Infiltration, percolation to ground water storage, and evaporation are all considered Initial Loss ‘\(L_i\)’ in Mm (USACE, 2000b).

2.6. Direct Runoff parameter

The simulation of rainfall-runoff modeling with HEC-HMS produces an initial abstraction and direct runoff. The HEC-HMS model permits direct runoff using six techniques. In the current study, we selected the SCS unit hydrograph technique to estimate the direct runoff component from separate storm events [29-32]. This method is intended for modeling watersheds with highly variable land uses and land covers [29], which influence overall basin storage.

Discharge from the catchment during a period, \(t\) is presented in Equation 3 and Equation 4 [33].

\[
Q_t = C_AI_t + C_BQ_{t-1} \tag{3}
\]

Where \(I_t\) is the average inflow to storage at time \(t\), and \(C_A\) and \(C_B\) are routing coefficients given by:

\[
C_A = \frac{\Delta t}{S_t + 0.5\Delta t} \quad \text{and} \quad C_B = 1 - C_A \tag{4}
\]

Where; \(\Delta t\) represents the computational time stage.

3. Results and Discussions

The sensitivity procedure described in the previous section has been applied to the Morai watershed using precipitation data of the year 2010—a historical flood year in Pakistan. The CNs have been assessed, ranging from 40-90, for their impact on initial loss and direct runoff. The changing CN displayed a dynamic effect on the outputs. The results are discussed in detail here.

![Sensitivity analysis of discharge v/s SCS Curve Number](image)

Figure 3. Curve numbers and their associated discharges

Figure 3 shows direct runoff for the event-based HEC-HMS model run for varying CNs for the 2010 flood event. It is known that direct runoff increases with the increase in CNs, but how much it changes is being ascertained in this study. The results depict that the maximum discharge of 362 cumecs was simulated using CN 90, while CN 40 produced a discharge of 78 cumecs keeping all other model parameters unchanged.
Figure 4. Sensitivity analysis of water availability

Figure 4 portrays the comparative analysis of initial abstraction and direct runoff in Million Acre Feet (MAF) by using CN values (40-90 with an increment of 5). Eleven CN values from 40-90 were used to simulate direct runoff. The results revealed that CN 40 generated 10,809.90 MAF of initial abstraction and produced 13,472.27 MAF of direct runoff. However, CN 90 simulated 1130.02 MAF of initial abstraction and 23,152.15 MAF of direct runoff.

Figure 5. Selection of SCS-CN and variation of water availability

Figure 5 illustrates the percent-wise variation of water availability using different CN values. Four groups with different CN ranges were selected to determine the range of simulated direct runoff within these groups. We tested the
CN groups with ranges 90-75, 75-70, and 90-40, which produced dynamic water availability within each group and reductions of 8.88%, 3.39%, 3.82%, and 41.81%, respectively. These variations are significant, and the range in resulting water availability highlights the importance of precise CN selection. The variability within each group is significant and indicates a high sensitivity of the model to this parameter, as also mentioned by Tassew et al. [17].

In systematic and proactive modeling approaches, numerous parameters are used to generate the quantitative figures, which are very problematic and challenging to acquire with accuracy [34]. However, in the current study, the sensitivity analysis justified the need for acquiring land use, land cover, and soil information to derive a precise CN in the study area. A small error in the selection of this parameter may result over- or underestimation of the flows, leading to severe consequences. Identifying the relationship between the model inputs and outputs is, therefore, an essential part of research [35]. Kousari et al. (2010) support the results achieved in the current study [36].

4. Conclusion

This study analyzes the sensitivity of the simulated runoff to CN. We conducted the analysis using the rainfall-runoff simulation in the HEC-HMS model by changing CN values and keeping all other parameters constant. We researched the ungauged Morai watershed, situated in a vulnerable region and droughts and floods have hit that. We used precipitation data from the flood year 2010 in the analysis to assess reliable water availability in the region. The year 2010 was a wet season, and even in the flood time, the model results show an alarming trend in the availability of water with changing CN values. Acquiring a precise CN value is imperative, since both over- and underestimation of flows may have serious consequences. Underestimation during a flood time may lead to inadequate preparation for flood response. Overestimation during dry periods may cause water and food shortages. Both water excess and deficit may become disastrous; therefore, adequate water resource planning and management call for precise estimation of the flows. The margin of error needs to be minimized as much as possible, and this can only be done by first identifying the parameters that have the most influence on model results and then acquiring accurate values. In this study, CN has a significant quantitative impact on direct runoff, the accuracy of which is essential for the estimation of floods during wet periods and water availability during dry seasons.

5. Acknowledgements

This research was conducted, and the current manuscript was prepared, using the GIS and Computer Labs of the U.S.-Pakistan Center for Advanced Studies in Water (USPCAS-W) / Mehran University of Engineering & Technology, Jamshoro. With great pleasure, the authors acknowledge the support provided by the FBLN/MetaMeta under “Africa to Asia and Back: Testing Adaptation in Flood Based Farming Systems.” Cooperation and supervision of the faculty of the USPCAS-W. Last but not least, many thanks to USAID for providing the technical support for the establishment of the Center.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Structural Behavior of High Strength Laced Reinforced Concrete One Way Slab Exposed to Fire Flame

Anas Ibrahim Abdullah a*

* Department Civil Engineering, College of Engineering, University of Baghdad, Baghdad, Iraq.

Received 31 July 2019; Accepted 18 October 2019

Abstract

In this study, an experimental investigation had conducted for six high strength laced reinforced concrete one-way slabs to discover the behavior of laced structural members after being exposed to fire flame (high temperature). Self-compacted concrete (SCC) had used to achieve easy casting and high strength concrete. All the adopted specimens were identical in their compressive strength of ($f'_c \approx 60$ MPa), geometric layout 2000×750×150 mm and reinforcement specifics except those of lacing steel content, three ratios of laced steel reinforcement of (0.0021, 0.0040 and 0.0060) were adopted. Three specimens were fired with a steady state temperature of 500℃ for two hours duration and then after the specimens were cooled suddenly by spraying water. The simply supported slabs were tested for flexure behavior with two line loads applied in the middle third of the slab (four-point bending test). The average residual percentage of cubic compression strength and splitting tensile strength were 57.5% and 50% respectively. The outcomes indicated that the residual bending strength of the burned slabs with laced ratios (0.0021, 0.004, 0.006) were (72.56, 70.54 and 70.82%) respectively. However; an increase in the deflection was gained to be (11.34, 14.67 and 17.22%) respectively with respect to non-burned specimens.

Keywords: Laced Reinforced Concrete; One-Way Slab; Fire Flame; High Temperature; SCC.

1. Introduction

Normal reinforced concrete (NRC) is known to have bounded ductility and confinement of concrete; NRC can be enhanced by appropriate amendment in materials of concrete and by considering suitable alteration in the reinforced details. Laced bars are reinforcing bars that extend in a direction parallel to the main reinforcement, they are bending into a diagonal manner between top and bottom reinforcement. Laced bars usually enclose temperature reinforcement bars which are placed outside the main reinforcement. Laced member is reinforced with two identical mats of steel bars for tension and compression. Through truss behavior, laced bars tied the two principal reinforcement and bounded the concrete between the reinforcement by truss action, they are also placed in a reciprocal way to encompass all transverse reinforcement as shown in Figure 1. Laced reinforcement for concrete members improves the ductility and produces better confinement for concrete [1]. Recently, considerable researches have studied the behavior of laced element under static and dynamic loads, in addition the use of laced reinforcement for blast resistant structures that leads to an urgent need to study the behavior of laced reinforced members exposed to fire flame.

Slab members have a significant effect by fire because of its large surface that exposed to fire relatively to its depth. Besides, fire may be exposed from one side of the member; which produced a gradation in temperature over slab thickness. An experimental program was carried out by Moss et al. (2008) [2], to study the behavior of two way reinforced concrete slab affected by fire. Moss concluded that concrete and reinforcement on the bottom of the slab were
heated well prior the top reinforcement and concrete. When the main reinforcement in the bottom reached 300°C, the yield strength of the main reinforcement started to decrease, that’s way bending moment and membrane strength decrease. Ghoreish et al. (2010) [3], indicated that the existence of imposed load during the burning process lead to a bad effect on slab behavior. Harada et al. (1972) [4], found that the residual compressive strength of concrete at a temperature 300°C to reference specimen was 60%, and the residual of steel concrete bond was 44%. Experimental test was done by Izzat et al. (2012) [5] to investigate the behavior of one-way reinforced SCC slab under fire flame effect, concluded that the residual flexural strength of the slab that cooled gradually was (81.5%, 75%, 62.3%) for fire temperature (300°C, 500°C, 700°C) respectively. Also, it was concluded that increasing the compressive strength decrease the residual flexural strength percent and sudden cooling is more effective to the residual flexural strength than gradual cooling while the deflection was increased with the increasing of burning temperature. Another study presented by Mohammed and Fawzi (2015) [6] investigated the structural behavior of SCC beams under the effect of repeated load after being exposed to fire flame of steady state temperature (200,300,400 and 500) °C and two different methods of cooling, sudden and gradual. The results showed that number of cycles that’s required to vanish the residual deflection was directly proportion with the burning temperature and sudden cooling method. It was also noticed that the failure mode changed to be combined shear-flexure instead of pure flexure due to the drop of the compressive strength amount.

The main purpose of using shear reinforcement is to enhance the behavior of structural members in large deflection stage by connecting the two main reinforcement. In the design of traditional structures, the essential purpose of shear reinforcement is to prohibit formation and spread of inclined tension cracks [7]. A wide range of experimental investigations conducted on (RC) and (LRC) beams by Parameswaran et al. (1986) [8], indicated that the support rotation angles range from 3.5° to 8°. The extended laced steel bars inclined from the horizontal plane at an angle 45° and 60°. The main objective of the investigations is the using laced steel reinforcement leads to improve the ductility and the load carrying capacity the beams compared with the beams traditional shear reinforcement.

A test study was executed by Keshava Rao et al. (1992) [9] to investigate the effect of blast load on the laced reinforced concrete members. It was concluded that the laced reinforcement increases the strength of the member by 25% under blast loading. A test programme presents by Akshaya et al. (2015) [10] to investigate the effect of monotonic and cyclic loads on the behaviour of laced steel-concrete composite beams with 60° lacing with and without fiber (LSCC) and (FLSCC). Their results revealed that load carrying capacity and ductility index for (FLSCC) beams was higher than (LSCC) and RC beams. The increasing in load carrying capacity was about 46% and 22% for (FLSCC) and (LSCC) respectively with respect to RC beams.

Experimental study was carried out by Allawi and Jabir (2016) [11] to study the behavior of one-way laced reinforced concrete slab under static load. Nine specimens were test for flexural behavior with three parameters of laced steel ratio, tension steel ratio, and clear span to effective depth ratio (L/d). The results indicated that specimen with laced steel ratio of (0.0065) gives an increase in ultimate load by about 57% with respect to specimen without laced ratio, also the decreasing (L/d) ratio by (31.25%) lead to increase failure load by about (103.57%) with respect to control specimen. Al-Ahmed and Hallawi (2019) [12], studied the influence of laced reinforcement on the behaviour of one way-slab under monotonic load. The test results showed that the cracking load and ultimate load increased by about (28% and 45%) and (16% and 40%) respectively for lacing ratio of (0.0026 and 0.0052) with respect to specimen without laced reinforcement. Also the ductility factor increased by about 33% and 49% for laced ratio 0.0026 and 0.0052 with respect to specimen without laced reinforcement.

2. Experimental Program

2.1. Materials

Ordinary Portland cement (Type-I), a natural sand of maximum size 4.87 mm, crushed gravel of (10mm) maximum size, tap water, a very fine pozzolanic material (Microsilica) as additives (pozzolan material) and ViscoCrete-5930 (product of Sika) were used in the adopted concrete mix to produce SCC for all the specimens. The results of sieve
analysis test for fine and coarse aggregate are presented in Table 1 and 2. The adopted concrete mix was designed according to EFNARC (2005) [13] to satisfy fresh properties of SCC and to match the expected compressive strength. Materials proportion for SCC are shown in Table 3, and tests for fresh SCC were complied with the limit of EFNARC (2005) [13] as shown in Table 4.

### Table 1. Grading of the fine aggregate

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Passing by Weight %</th>
<th>Cumulative Passing % Limit of Iraqi Specification No. 45/1993</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Zone 1 Zone 2 Zone 3 Zone 4</td>
</tr>
<tr>
<td>10</td>
<td>100</td>
<td>100 100 100 100</td>
</tr>
<tr>
<td>4.75</td>
<td>91</td>
<td>90-100 90-100 90-100 95-100</td>
</tr>
<tr>
<td>2.36</td>
<td>75.5</td>
<td>60-95 75-100 85-100 95-100</td>
</tr>
<tr>
<td>1.18</td>
<td>56.5</td>
<td>30-70 55-90 75-100 90-100</td>
</tr>
<tr>
<td>0.60</td>
<td>39.4</td>
<td>15-34 35-59 60-79 80-100</td>
</tr>
<tr>
<td>0.30</td>
<td>10.9</td>
<td>5-20 8-30 12-40 15-50</td>
</tr>
<tr>
<td>0.15</td>
<td>2.5</td>
<td>0-10 0-10 0-10 0-15</td>
</tr>
</tbody>
</table>

### Table 2. Grading of the coarse aggregate

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Cumulative Passing % Limit of Iraqi Specification No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.5</td>
<td>100 100</td>
</tr>
<tr>
<td>19</td>
<td>100 95-100</td>
</tr>
<tr>
<td>14</td>
<td>--- ---</td>
</tr>
<tr>
<td>9.5</td>
<td>95.7 30-60</td>
</tr>
<tr>
<td>4.75</td>
<td>5.6 0-10</td>
</tr>
</tbody>
</table>

### Table 3. Mix proportions of the used SCC mix

<table>
<thead>
<tr>
<th>Materials</th>
<th>Proportion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>600 (kg/m³)</td>
</tr>
<tr>
<td>Sand</td>
<td>760 (kg/m³)</td>
</tr>
<tr>
<td>Gravel</td>
<td>900 (kg/m³)</td>
</tr>
<tr>
<td>SF*</td>
<td>4%</td>
</tr>
<tr>
<td>w/b</td>
<td>0.27</td>
</tr>
<tr>
<td>SP**</td>
<td>2</td>
</tr>
</tbody>
</table>

* Replacement by weight of cement  
** Liter / 100 kg of cement

### Table 4. Tests for Fresh SCC

<table>
<thead>
<tr>
<th>Property Measured</th>
<th>Test Method</th>
<th>Test Values</th>
<th>EFNARC Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flowability</td>
<td>Slump Flow</td>
<td>690 mm</td>
<td>600-850 mm</td>
</tr>
<tr>
<td>Flowability</td>
<td>T₅₀₀</td>
<td>4 Sec</td>
<td>2-5 Sec</td>
</tr>
<tr>
<td>Passing Ability</td>
<td>L-Box</td>
<td>0.84</td>
<td>≥ 0.75</td>
</tr>
</tbody>
</table>

Figure 2. Fresh concrete tests
2.2. Test Specimens

Six simply supported laced reinforced SCC one-way slabs were cast and cured. All the specimens had the same geometrical layout, compressive strength and reinforcement specifics except laced steel ratio. The specimens were designed according to ACI 318M-2014 [14] and accepted with UFC 3-340-02, 2008 [1], for laced reinforcement details. The specimens had divided into three pairs, each pair had different laced steel ratio (0.0021, 0.0040 and 0.0060). For each pair, one slab was burned with steady state fire temperature 500°C for a duration of two hours and the other was not. The considered cooling method was sudden cooling by spraying water. Then after, all specimens had tested under monotonic load of two parallel line load till the failure. The slab details and dimensions are shown in Figure 3. All specimens had the same reinforcement for compression and tension of 8 mm diameter deformed rebar at 100 mm c/c spacing and temperature reinforcement of 8 mm diameter deformed rebar at 120 mm c/c spacing for top and bottom. The characteristics of the tested slabs are illustrated in Table 5.

Figure 4 shows the positions of the strain and dial gauges. Two dial gauges were used, one was installed in the central of the slab and the other near to support. Three strain gauges (5 mm length) were installed in tension, compression, and lacing steel bars in the mid span. Two strain gauges (30 mm) were fixed at top and bottom concrete face in mid span. The used strain gauges were of (120Ω) resistance made in japan for TML. Strain gauges for steel bar hadn’t used for burning specimens because it will be destroyed by fire. An instrument consists of thirteen metal plates of different thickness ranging from 0.05 to 1.0 mm was used to measure the width of cracks. Thermocouple was used for measuring the temperature of the furnace and the specimen. Dial gauge was used during the burning process to measure the central deflection of slabs. Figure 5 shows the testing rig and specimen position.

![Diagram](image_url)

**Figure 3.** Details and dimensions of test slab specimens

![Diagram](image_url)

**Figure 4.** Instrumentation detailed and position

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimens Designation</th>
<th>Tension steel ratio ($\rho_t$)</th>
<th>Laced steel ratio ($\rho_s$)</th>
<th>Laced angle ($\theta$)</th>
<th>Temp. ($^\circ$C)</th>
<th>Laced steel details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S42/21NB</td>
<td>0.0042</td>
<td>0.0021</td>
<td>45°</td>
<td>30</td>
<td>Ø6 mm at 110 mm c/c</td>
</tr>
<tr>
<td>2</td>
<td>S42/40NB</td>
<td>0.0042</td>
<td>0.0040</td>
<td>45°</td>
<td>30</td>
<td>Ø6 mm at 60 mm c/c</td>
</tr>
<tr>
<td>3</td>
<td>S42/60NB</td>
<td>0.0042</td>
<td>0.0060</td>
<td>45°</td>
<td>30</td>
<td>Ø8 mm at 70 mm c/c</td>
</tr>
<tr>
<td>4</td>
<td>S42/21B</td>
<td>0.0042</td>
<td>0.0021</td>
<td>45°</td>
<td>500</td>
<td>Ø6 mm at 110 mm c/c</td>
</tr>
<tr>
<td>5</td>
<td>S42/40B</td>
<td>0.0042</td>
<td>0.0040</td>
<td>45°</td>
<td>500</td>
<td>Ø6 mm at 60 mm c/c</td>
</tr>
<tr>
<td>6</td>
<td>S42/60B</td>
<td>0.0042</td>
<td>0.0060</td>
<td>45°</td>
<td>500</td>
<td>Ø8 mm at 70 mm c/c</td>
</tr>
</tbody>
</table>
2.3. Burning and Cooling

Three specimens were burnt in a furnace made for this purpose with fire flame. The specimens were installed on two supports in the furnace and a uniform load was applied to the specimen of 10.64 kN/m². Blocks of net mass around 50 kg were distributed uniformly on slab to obtain uniform load during the fire test duration. Burning test was according to ASTM E119 (2000) [15]. The specimens were burned for two hours of steady state temperature 500°C. The required time to reach 500°C was 5 minutes. Both deflection and specimen’s temperature were recorded during the fire test duration. The cooling method was sudden cooling by spraying water to slab after burning time. The burning process details were the same for the three slabs. Figure 6 shows the furnace and the slab position.
3. Results And Discussions

3.1. Non-burned Specimens

The experimental program included testing three reinforced concrete one-way slabs that non-burned to study the effect of laced reinforcement on one-way slab. The mode of failure for all the tested slabs were flexure. The first flexural crack was firstly appeared in the middle third of the tension face for the slab where maximum moment occurred. As the load increased further, additional flexural cracks were generated and extended in the bottom surface of the slab parallel to the initial crack and the supports direction. Then, the cracks were grown to the sides of the slab and reached to the top edge of the slab at failure stage. There was noticed that the cracks located in the middle third of the slab and no cracks were observed near the supports. Eventually, the failure of the specimens happened due to the extensive yield of tensile steel bars. The test results regarding the initial crack loads and ultimate loads are illustrated in Table 6.

Table 6. Cracking and ultimate loads for non-burned specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Ultimate Load (Pu) (kN)</th>
<th>First Crack Load (Pcr) (kN)</th>
<th>Load at the (0.4mm) Crack Width (kN)</th>
<th>% Increase in (0.4mm) Cracking Load with Respect to Ref.</th>
<th>Pcr/Pu</th>
<th>% Increase in Ultimate Load with Respect to Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>S42/21NB</td>
<td>143.22</td>
<td>33.35</td>
<td>89.6</td>
<td>Ref.</td>
<td>23.29</td>
<td>Ref.</td>
</tr>
<tr>
<td>S42/40NB</td>
<td>155.98</td>
<td>33.35</td>
<td>92</td>
<td>2.67</td>
<td>21.38</td>
<td>8.9</td>
</tr>
<tr>
<td>S42/60NB</td>
<td>177.56</td>
<td>33.35</td>
<td>116.24</td>
<td>29.73</td>
<td>18.78</td>
<td>23.97</td>
</tr>
</tbody>
</table>

The first crack occurred for all specimens that non-burned in the same load (33.35 kN) that’s belong to the similarity in the characteristics of compressive strength and main reinforcement ratio and also indicated that increasing laced reinforcement do not effect on the first cracking load. Cracks width behavior and load increments are shown in Figure 7: it can be detected from this figure that the increasing rate of crack width is highly sensitive after yielding stage. Generally, it is obvious from the results that increasing laced steel ratio increased the failure load, the increasing was (8.9%, 23.97%) for specimens (S42/40NB, and S42/60NB) with respect to the specimen (S42/21NB). The deflection for specimens were discussed at service load and ultimate load. Tan and Zhao, 2004 [13], indicated that service load is about (70-75%) of the ultimate load. In this study the service load was taken to be 70% of the ultimate load. The results of this study show that, the increasing of the laced ratio is directly proportion with the ultimate deflection as shown in Figure 8, also from this figure the load-deflection curve for the non-burned specimens had the same behaviour and diverge after yielding of tension reinforcement. The increasing in the deflection at ultimate load was (5.03% and 21.7%) for specimens (S42/40NB and S42/60NB) with respect to (S42/21NB).
The central deflections at ultimate and service loads are given in Table 7. Increasing laced steel ratio decreases the deflection at ultimate load of reference specimen about 22% and 42% for specimens (S42/40NB and S42/60NB) with respect to (S42/21NB). The absorbed deflection-load curves were matched up to yield stage then after they spaced to produce a different level of absorbed energy to be (6021, 6887, 9133) for the specimens (S42/20NB, S42/40NB and S42/60NB) respectively. Ductility factor was calculated for all the specimens as illustrated in Table 6, which is the rate of central deflection at failure load to the central deflection at yield point. It was obvious from this table that increasing laced steel ratio increases the ductility factor. Load-strain behavior for laced bars is shown in Figure 9, which illustrates that small strains were recorded in the first but after yielding of the tension steel bars the strain increasing rapidly, also it can be noted that the yielding of the central laced bar at failure stage. Also load-strain for tension steel bar and top fiber of concrete was recorded as shown in Figures 10 and 11. In general, it is obvious that the influence of laced reinforcement was to prevent the tension reinforcement strain during its strain hardening region. At failure the strain at concrete top fiber was between (4438 and 3332) micro-strain.

Figure 8. Effect of lacing reinforcement on load-deflection behavior

Table 7. Central deflection at service and ultimate loads for non-burned specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Deflection at Service Load (mm)</th>
<th>% Decrease in Deflection at Service Load</th>
<th>Deflection at Ultimate Load of Ref. Specimen</th>
<th>% Decrease in Deflection at the Ultimate Load of Ref. Specimen</th>
<th>Ultimate Deflection (mm)</th>
<th>% Increase in Deflection at Ultimate Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>S42/21NB</td>
<td>17.48</td>
<td>Ref.</td>
<td>55.99</td>
<td>Ref.</td>
<td>55.99</td>
<td>Ref.</td>
</tr>
<tr>
<td>S42/40NB</td>
<td>18.51</td>
<td>5.6</td>
<td>43.66</td>
<td>22.02</td>
<td>58.81</td>
<td>5.03</td>
</tr>
<tr>
<td>S42/60NB</td>
<td>21.94</td>
<td>25.5</td>
<td>32.45</td>
<td>42</td>
<td>68.14</td>
<td>21.7</td>
</tr>
</tbody>
</table>
Figure 9. Effect of lacing steel ratio on lacing strain behavior for non-burned specimens

Figure 10. Effect of lacing steel ratio on the tension steel strain of non-burned specimens

Figure 11. Effect of lacing steel ratio on concrete compression strain
3.2. Burned Specimens

The results for burned specimens indicated that cracks were spread on the surfaces of the slabs after firing and cooling process, also flexural cracks were appearing due to fire and imposed load in the bottom and sides of the slabs. Concrete compressive strength for testing cubes after burning and cooling showed that the compressive strength is decreased. The residual compressive strength was (57%, 59%, 57%) for specimens (S42/21B, S42/40B and S42/60B) respectively. Also the residual splitting tensile strength for testing cylinders after burning and cooling was (50%, 52%, 49%) for specimens (S42/21B, S42/40B and S42/60B) respectively. The deflection of the specimens during the burning process increased at a faster rate in the first twenty minutes, after that the deflection approximately remains constant and returned to decrease in the second hour of burning as shown in Figure 12. After completion the burning and cooling process the specimens were tested under static load of two line loads. Flexural failure was occurred for all the slabs. Firstly, the cracks were generated at the bottom of the slab (new cracks), also, grew from fire cracks in the bottom of the slab. The crack width-load behavior is shown in Figure 13. The outcomes indicated that the deflection and load at the failure stage increase as laced reinforcement is increased, Table 8. And the deflection behavior is shown in Figure 14. Also strain-load behavior for concrete compression face for burned slabs are shown in Figure 15.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Ultimate Deflection (mm)</th>
<th>Ultimate load (kN)</th>
<th>% Increase in ultimate deflection</th>
<th>% Increase in ultimate load</th>
</tr>
</thead>
<tbody>
<tr>
<td>S42/21B</td>
<td>62.34</td>
<td>103.92</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S42/40B</td>
<td>68.92</td>
<td>110.03</td>
<td>5.03</td>
<td>5.88</td>
</tr>
<tr>
<td>S42/60B</td>
<td>82.32</td>
<td>125.76</td>
<td>21.89</td>
<td>21.02</td>
</tr>
</tbody>
</table>

Figure 12. Central deflection-time history for burned specimens for non-burned specimens

Figure 13. Load–Strain Relation for Concrete Top Surface at Mid–span for Burned Specimens
3.3. Comparison between Burned and Non-Burned Specimens

Cracks width behavior for the burned and non-burned specimens are shown in Figures 16 to 18. It is clear that crack width-load curves for burn and non-burn state had two stages which represents two intervals (before and after yield stage). The results also indicated that cracks pattern are similar for all the tested specimens as shown in Figure 19. Flexural failure was occurred for all the specimens due to the yield of the tension reinforcement and excessive deflection. The failure loads for all the tested slabs in this study are given in Table 9. It is obvious that fire decreases the ultimate load of the slab, the residual ultimate loads were (72.56, 70.54 and 70.82%) for specimens (S42/21B, S42/40B and S42/60B) with respect to reference specimens. The ultimate deflection increases for specimens affected by fire with respect to non-burned specimens, the increasing was (11.34, 14.67 and 17.22%) for specimens (S42/21B, S42/40B and S42/60B) respectively with respect to the reference specimens as shown in Figure 20. The recorded strain-load for concrete compression top fiber indicated that the strain at failure stage was (5270, 4518) microstrain for burned specimens as shown in Figure 21. It is more than the strain recorded for non-burned specimens. Concrete crushing at failure load was happened to burned specimens while there was no crushing observed for non-burned specimens. Table 10 illustrates the results for all specimens.
Table 9. Ductility factor for the non-burned specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Steel Yielding Load (kN)</th>
<th>Yield Deflection (mm)</th>
<th>Ultimate Deflection (mm)</th>
<th>Ductility Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>S42/21NB</td>
<td>82.17</td>
<td>10.12</td>
<td>55.99</td>
<td>5.53</td>
</tr>
<tr>
<td>S42/40NB</td>
<td>85.33</td>
<td>9.46</td>
<td>58.81</td>
<td>6.22</td>
</tr>
<tr>
<td>S42/60NB</td>
<td>92.5</td>
<td>9.83</td>
<td>68.14</td>
<td>6.93</td>
</tr>
</tbody>
</table>

Figure 16. Influence of Burning and Cooling on the Cracking Behavior for Specimen with Laced Ratio (0.0060)

Figure 17. Influence of Burning and Cooling on the Cracking Behavior for Specimen with Laced Ratio (0.0040)
Figure 18. Influence of Burning and Cooling on the Cracking Behavior for Specimen with Laced Ratio (0.0021)

Figure 19. Cracks pattern for burned and non-burned specimens
Table 10. Data for burned and non-burned specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Ultimate Load ($P_u$) (kN)</th>
<th>Ultimate Deflection (mm)</th>
<th>First Crack load ($P_{cr}$) (kN)</th>
<th>Residual strength</th>
<th>Compressive Strength (MPa) at 60 days</th>
<th>Inc. in ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>S42/21NB</td>
<td>143.22</td>
<td>55.99</td>
<td>33.35</td>
<td>-</td>
<td>58.66</td>
<td>-</td>
</tr>
<tr>
<td>S42/40NB</td>
<td>155.98</td>
<td>58.81</td>
<td>33.35</td>
<td>-</td>
<td>61.56</td>
<td>-</td>
</tr>
<tr>
<td>S42/60NB</td>
<td>177.56</td>
<td>68.14</td>
<td>33.35</td>
<td>-</td>
<td>58.03</td>
<td>-</td>
</tr>
<tr>
<td>S42/21B</td>
<td>103.92</td>
<td>62.34</td>
<td>Precracking</td>
<td>72.56%</td>
<td>33.5</td>
<td>11.34%</td>
</tr>
<tr>
<td>S42/40B</td>
<td>110.03</td>
<td>68.92</td>
<td>Precracking</td>
<td>70.54%</td>
<td>30.75</td>
<td>14.67%</td>
</tr>
<tr>
<td>S42/60B</td>
<td>125.76</td>
<td>82.32</td>
<td>Precracking</td>
<td>70.82%</td>
<td>30.39</td>
<td>17.22%</td>
</tr>
</tbody>
</table>

Figure 20. Central deflection-load behavior for burned and non-burned specimens

Figure 21. Load-strain behavior of concrete top fiber for burned and non-burned specimens
4. Conclusion

A test program was performed for six simply supported high strength reinforced concrete one-way slabs with reciprocal laced steel bars. As foresaw that flexural failure was occurred for all specimens by excessive deflection and the yield of tension steel bars, the flexural cracks for static test were located in the middle third of the bottom face for the slabs (constant moment). The cracks appeared after burning the specimens were distributed on the surface of the slab and approximately no cover spalling, also the deflection-time history of burned specimens indicated that the deflection decreased in the second hour. Increasing laced ratio for burned and non-burned specimens leads to an increasing in the failure load and ductility factor of the specimens, also the ultimate deflection and service deflection were increased. Load-strain curves improved that laced steel bars restricted tension reinforcement to strain in the strain hardening region, however concrete strain of the compression face was increased in a non-linear manner with load till the fail of the slab.

The compressive strength of concrete after exposure to fire flame was decreased, for 500°C burning and sudden cooling the residual compressive strength percent was approximately (57.5%). Also, the failure load was decreased by about (28.7%). While the ultimate deflection was increased by about (14.41%).

5. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Mineralogy, Micro-fabric and the Behavior of the Completely Decomposed Granite Soils

Elsayed Elkamhawy a, b, Bo Zhou a, Huabin Wang a*

a School of Civil Engineering and Mechanics, Huazhong University of Science and Technology, Wuhan, Post code 430074, P.R. China.
b Faculty of Engineering, Zagazig University, Zagazig, Post code 44519, Egypt.

Received 16 September 2019; Accepted 20 November 2019

Abstract

The main objective of this study is to investigate the impact of the micro-fabric and soil mineralogy on the overall macro-behavior of the completely decomposed granite soil through a set of drained and undrained triaxial shearing and isotropic compression tests on a medium-coarse grading completely decomposed granite soil. The mineral composition of the soil was a substantial factor governing the compressive behavior. The soil compressibility increased significantly in the case of existence crushable and weak minerals within the soil minerals like fragile feldspar, as well as the high content of fines, especially the plastic fines. The scanning electron microscopic photos indicated that the micro-fabric of the soil had a paramount impact on the compressive behavior. The mechanism of the volumetric change depended on the stress levels, the soil mineral composition and the grain morphology. In the low consolidated stress levels, the soils’ grains rearrangement was the prevailing mechanism of the volumetric change, particularly with the absence of weak and crushable minerals. On the other hand, at the high consolidated stress levels, particles’ crushing was the prevailing mechanism in the volumetric change. Both the mechanisms of volume change could occur simultaneously at the low stress levels in the case of presence crushable minerals in addition to micro-cracks in the soil grains. The soil showed an isotropic response after 250 kPa, as this stress level erased the induced anisotropy from the moist tamping preparation method. Under the drained shearing conditions, the soil showed a contractive response, while during the undrained shearing conditions, the soil exhibited both the contractive and dilative responses with phase transformation points. The studied soil showed a unique critical state line, irrespective of the drainage conditions and initial states, the critical state line was parallel to the isotropic compression line in the void ratio—effective stress space. In the deviator—effective mean stresses space, the studied soil approached a unique CSL with a critical stress ratio equal 1.5, corresponding to critical friction angle of 36.8°.

Keywords: Completely Decomposed Granite; Soil Mineralogy; Micro-fabric.

1. Introduction

Because of the abundance of the completely decomposed granite (CDG) soil in many countries around the world, it is vastly utilized in engineering practices for example: back-fill materials for retaining walls, construction of earth dams, embankments of waterways, and roads. Behavior of residual soils is completely different from sedimentary soils. Residual soils such as granitic saprolite soils originated from the chemical and physical weathering processes of granite rock, thus the mineralogy and micro-fabric of the parent rock are considered substantial factors governing the resulted soils behavior [1]. Residual soils are generally well-graded and include a broad domain of grain size distributions [2]. Many studies have shown that the grading of granitic saprolite soil depends on the environmental circumstances of the
weathering process, mineralogy and micro-fabric of the parent rock; in which the soil grading curve shifts towards the fine domain as the weathering grade increases [3-6].

On account of shortage and insufficient data available about mineral composition of soils in addition to grains morphology, researchers have concluded that grading is considered the dominant factor in the compressive behavior of saprolitic soils [2, 7]. Ham, Nakata [8] found that content of gravel had a significant impact on the compression characteristics of CDG soils. Ham, Nakata [9] have shown that the compressibility of CDG soil is robustly affected by the strength of single particle. Studies conducted on CDG soils have shown that the uniqueness of isotropic compression lines (ICLs) is a general feature of such these kinds of soil [10-12]. However, nonuniqueness of ICLs was observed by Wang and Yan [13], as the ICLs were nearly parallel; such behavior is termed “transitional behavior”, which could be attributed to the soil fabric. Wang and Yan [13] investigated undisturbed soil, while, re-compacted samples have been reported by Yan and Li [12], Ng, Fung [11], and Lee and Coop [10]. Therefore, the impact of the parent rock fabric is clearly pronounced in the intact soil behavior. The non-uniqueness of ICLs was also reported by Elkamhawy, Zhou [14] in the reconstituted CDG soil by increasing sand contents, thus the induced fabric by increasing sand content withstood during the isotropic compression path.

Unique critical state lines (CSLs) were also found in the deviator and effective mean stresses q-p' space and in the e-\log p' space, regardless of the drainage conditions and initial states. Although the reconstituted CDG soil and intact soil samples reported by Elkamhawy, Zhou [14] and Wang and Yan [13], respectively exhibited nonuniqueness of the ICLs, unique CSLs were also found in the e-\log p' space. This could be explained as follows; whatever the fabric triggered from the isotropic compression stage, it was not sturdy enough to persist during shearing, and it was completely destroyed during large strain-shearing path, leading eventually to unique CSL.

To meet the rapid development in china, granitic saprolites are extensively used in many engineering applications. Because of the shortage in studies that dealt with these soils, the behavior is not well understood and many engineering problems occurred, in addition to the fact that the weathered rocks get involved in various modes of landslides [15, 16] leading to loss of lives and property. Thus there is urgent need for further studies to understand the mechanics of this kind of soil. Therefore, the main objectives of the current paper are providing well understanding of the CDG soil behavior and illuminating the influences of the micro-fabric and mineralogy on the mechanical characteristics via a set of drained and undrained triaxial compression shearing and isotropic compression tests performed on medium-coarse CDG soil originated from the weathering of the biotite granite rock. The scanning electron microscopic photos (SEM) and X-ray diffraction analysis (XRD) have been employed to interpret the experimental results. The presented results will be more helpful for developing an advanced constitutive model for the saturated remolded CDG soils.

2. Research Methodology and Material Characteristics

This section provides the research methodology as indicated in Figure 1. The studied CDG soil in this paper was brought from Gaoliang city, Guangdong province, southeast China. Preliminary tests such as soil grading, specific gravity, Atterberg limits, maximum dry density, X-ray diffraction analysis, and scanning electron microscopic tests were then implemented on the studied CDG soil. After that the triaxial samples were prepared using the moist tamping technique. The soil samples were then installed in the triaxial device for saturation, consolidation, and shearing tests. The following sections indicate in detail the soil characteristics and the testing program step by step.

Figure 1. The flow chart of the research methodology
2.1. Physical Characteristics

Based on the classification system of the Geotechnical Engineering Office [17], the studied soil was completely decomposed granite (i.e., grade V). Grading curve of the CDG soil is shown in Figure 2, in addition to other reported data. It can be clearly seen that the soil is well-graded such as other CDG soils in South Korea [10] and Hong Kong [11, 12]. It can be also concluded that the environmental circumstances of the weathering process of the studied soil and the soil in Hong Kong were close, as the soils have almost close grading. The soil is classified as low plasticity clayey sand (SC-CL) based on the unified soil classification system (USCS). The maximum dry density was 1810 kg/m3 at the optimum water content 15.2%. The soil specific gravity was 2.58. Plastic and liquid limits have been obtained from the fall cone test on the particles passed from No. 40 sieve (425 μm) that were 24.65% and 39.85%, respectively.

Figure 2. Soil grading

2.2. Mineralogical Features

By visual inspection, the CDG soil was yellowish brown with black and white spots, the geological map indicates that the parent granite rock of the CDG soil is biotite granite, as illustrated in Figure 3. In the soil site, there are no core stones, meaning that the parent biotite rock decomposed totally [18].

The X-ray diffractometer (XRD) analysis was conducted on the CDG soil samples, the XRD results were compared to the corresponding spectra for the bulk specimens of CDG. The XRD analysis showed that the soil minerals included quartz, sodium and potassium feldspar, calcite, dolomite, and clay in the form of illite and kaolinite minerals. Presence of kaolinite mineral shows well-drained conditions during the chemical weathering process. The small amounts of illite group can be attributed to the limited amounts of cations removed in a stagnant state at the slope bottom [4]. Table 1 gives the statistical analysis for the composition of the soil minerals. Quartz was found to be the most abundant mineral; this could be explained by the fact that it is inert to the chemical weathering and is considered the most stable mineral. The inert and stable minerals can be arranged starting with quartz followed by muscovite then alkali feldspar and finally plagioclase feldspar, which is considered the most mineral prone to the chemical weathering, forming finally clay minerals [19, 20]. Accordingly, the content of potassium feldspar was higher than sodium feldspar.
2.3. Test Program and Samples Preparation

The drained and undrained triaxial shearing and isotropic compression tests were performed with an automated triaxial stress path test apparatus produced in the UK by Global Digital Systems Ltd. (GDS). The device is dependent on the classic Bishop and Wesley stress path triaxial cell, so that control directly in stresses on the soil specimen. The GDS triaxial device was provided with axial displacement controller, back and cell pressure controllers as well as a linear variable displacement transducer (LVDT) for the axial strain measurement. The GDS device permits the water drainage from the upper platen, the volume changes of the soil samples were measured through the back-pressure controller, and the pore water pressure is measured from the lower platen.

The soil sample was prepared by the moist tamping technique, as the soil was firstly oven dried and to avoid using large samples, the soil sieved through a No. 4 sieve. The sieved soil was then remixed with water at the optimum water content. After that the soil was compacted inside a split mold in three layers, with scratching between the neighboring surface layers for decreasing layering effect. By controlling the applied compaction effort and the soil weight in each layer, different initial densities were obtained. The split mold was disassembled after compaction; the soil sample was then encased by the rubber membrane. It is worth to note that the sample can stand freely after the split mold was detached and before encapsulation with the rubber membrane due to the induced suction forces, as the soil sample is partially saturated. The sample was then immersed in a tank of water under a vacuum pressure of 0.1 MPa for 24 h to increase the degree of saturation. After the preliminary saturation and prior to install sample in the GDS device, the

---

Table 1. Minerals composition of the CDG soil

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Mean</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Illite</td>
<td>8.4</td>
<td>5</td>
<td>13</td>
<td>8</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>15.8</td>
<td>3</td>
<td>22</td>
<td>19</td>
</tr>
<tr>
<td>Quartz</td>
<td>46.3</td>
<td>26</td>
<td>62</td>
<td>36</td>
</tr>
<tr>
<td>Potassium feldspar</td>
<td>27.9</td>
<td>19</td>
<td>52</td>
<td>33</td>
</tr>
<tr>
<td>Sodium feldspar</td>
<td>1</td>
<td>9</td>
<td>9</td>
<td>0</td>
</tr>
<tr>
<td>Dolomite</td>
<td>0.5</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Calcite</td>
<td>0.1</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

---
sample dimensions were then measured using a Vernier caliper with 0.01 mm resolution, as the diameter was measured in top, bottom and middle of the sample and taken the average value, the sample height was also measured in three positions with an angle 120° and taken the average value. The soil specimen was then combined in the triaxial apparatus with assembling all required connections. To preserve a fixed effective mean stress during the saturation process, the cell and back pressures were then increased together. To check the saturation degree, the saturation check was implemented by measuring the B-value proposed by Skempton [21], which was 0.97 for all soil samples.

During the isotropic compression test, the rates of loading and unloading were 15 kPa/h and 30 kPa/h, respectively. The induced excess pore water pressure with using these rates was not remarkable. The triaxial compression shearing test was carried out through a strain-controlled mode. A 0.02%/min and 0.05%/min axial strain rates were employed for the drained and undrained shearing, respectively, derived from Head [22]. Tables 2–4 summarize the details of the isotropic compression and the triaxial compression shearing tests. Verdugo and Ishihara [23] discussed two methods to determine the void ratio of soil samples, in this study void ratio was measured using the second method via the back calculation of the final void ratio and the volumetric change with the assumption that the soil samples are totally saturated.

### Table 2. Testing conditions for the isotropic compression tests

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Compaction ratio</th>
<th>Loading path, P' kPa</th>
<th>Initial void ratio</th>
<th>Final void ratio</th>
<th>Slope of ICL(ξ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISO-85</td>
<td>85%</td>
<td>20-700</td>
<td>0.6500</td>
<td>0.4760</td>
<td></td>
</tr>
<tr>
<td>ISO-87.5</td>
<td>87.5%</td>
<td>20-250-150-450</td>
<td>0.6182</td>
<td>0.5045</td>
<td>0.06</td>
</tr>
<tr>
<td>ISO-90</td>
<td>90%</td>
<td>20-200-50-600</td>
<td>0.6011</td>
<td>0.5124</td>
<td></td>
</tr>
</tbody>
</table>

Note: ISO means isotropic compression and the numbers refer to the compaction ratio

### Table 3. Testing conditions for the drained triaxial shearing tests

<table>
<thead>
<tr>
<th>Test type</th>
<th>Compaction ratio, %</th>
<th>Consolidated stress P'(kPa)</th>
<th>Void ratio before shearing</th>
<th>Final void ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drained shearing (CD)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>85.65</td>
<td>100</td>
<td>0.5740</td>
<td>0.5074</td>
<td></td>
</tr>
<tr>
<td>85.40</td>
<td>200</td>
<td>0.5540</td>
<td>0.4721</td>
<td></td>
</tr>
<tr>
<td>85.42</td>
<td>300</td>
<td>0.5263</td>
<td>0.4488</td>
<td></td>
</tr>
<tr>
<td>87.68</td>
<td>400</td>
<td>0.5017</td>
<td>0.4318</td>
<td></td>
</tr>
</tbody>
</table>

### Table 4. Testing conditions for the triaxial undrained compression shearing tests

<table>
<thead>
<tr>
<th>Test type</th>
<th>Compaction ratio, %</th>
<th>Consolidated stress P'(kPa)</th>
<th>Final void ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undrained shearing (CU)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>88.45</td>
<td>100</td>
<td>0.5662</td>
<td></td>
</tr>
<tr>
<td>85.39</td>
<td>200</td>
<td>0.5354</td>
<td></td>
</tr>
<tr>
<td>87.04</td>
<td>300</td>
<td>0.5191</td>
<td></td>
</tr>
<tr>
<td>86.68</td>
<td>400</td>
<td>0.4958</td>
<td></td>
</tr>
</tbody>
</table>

### 3. Isotropic Compression

Isotropic compression response of the CDG soil is depicted in Figure 4. Irrespective of the initial void ratios, the ICLs combined in a unique compression line representing the soils’ state boundary surface. The unique ICL showed that the induced fabrics from the different compaction efforts were not strong enough and totally destroyed during the isotropic compression path and reached eventually a unique fabric at the end of the consolidation path. Comparing the studied soil with other reported CDG soils in Hong Kong [11, 12]. The studied soil showed low compressibility, where the gradient of the ICL (ξ) was 0.06, which was significantly lower than those in the CDG studied by Yan and Li [12], as ξ was 0.091. This can be illustrated from the grading curves, as the studied CDG soil was much coarser with less fines content. Although the CDG soil studied by Ng, Fung [11] almost has the identical coarse content with the studied soil, the slope of the ICL was high and ξ was 0.13, this can be attributed to the high fines content, in particular the plastic fines and presence of crushable feldspar.

The low compressibility of the CDG soil can be not only imputed to soil grading, but also to the soil mineralogy and micro-fabric. Although the grading of the CDG soil in South Korea reported by Lee and Coop [10] was coarser than the studied soil, the compressibility was higher than that in the studied soil, as ξ was 0.087. This can be explained by the soil mineralogy, micro-fabric and the stress levels. The minerals composition of the CDG soil in South Korea were feldspar 50%, quartz 33% with limited amounts of kaolinite 6%, mica 9% and 2% smectite. While, the minerals of the studied CDG soil were 46.3% quartz, 28.9% feldspar, 15.8% kaolinite, and 8.4% illite with small amounts of calcite 0.1% and dolomite 0.5%. It can be clearly seen that the dominant mineral in the studied CDG soil was quartz, whereas feldspar was the dominant one for the CDG soil in South Korea. Lee and Coop [10] studied the soil under stress levels
higher than those employed in this study, presence of mica also has a paramount influence on the compressibility of the soil even if exist by a limited portion [24]. Because of the presence of the crushable feldspar and the micro-cracks in the soils' grains as well grains morphology [25], the compressibility of the CDG soil in Hong Kong was higher than that in the studied soil. A scanning electron microscopic-photo illustrated in Figure 5 indicates the micro-fabric of the CDG soil. Clay minerals not only filled feldspar fissures and pores, but also were on feldspar surface; this means that clay minerals acted as a pillow for the larger-sized minerals, thus the compressibility of the CDG soil decreased substantially.

Figure 4. Isotropic compression response

Figure 5. Scanning electron microscopic-photo a) clay minerals in feldspar fissures, b) clay minerals on feldspar surface, and c) clay minerals filling pores
4. Volumetric Change

The soil specimens deformation was carefully examined during the isotropic compression and unloading stages. Figure 6 indicates the response of the soil samples, radial and axial strains were then plotted versus mean effective stresses. The soil response was represented by a 90% relative compaction. The soil exhibited an isotropic response during both the compression and relief paths. The volume change during unloading path was inconsiderable, thus volumetric change was unrecoverable. The axial strain was lower than the radial strain, which can be illustrated via the soil response. When the soil specimen is compacted in a specified direction, it becomes stiffer than other direction. Therefore, the contractive behavior becomes too low in this direction, while the response becomes more contractive and softer when the soil specimen is loaded in the lateral direction. These observations match closely with other CDG soils reported by Yan and Li [12] and Wang and Yan [13]. A 250 kPa effective mean stress was sufficient to erase the induced anisotropy from the compaction during the moist tamping method, in which both the axial and radial strain curves were quite parallel. Figure 7 also confirmed the isotropic response of the CDG soil, as after erasing the induced anisotropy from the preparation method, the slope of the relation of axial and radial strains was nearly 1:1. Since the dominant mineral was quartz and the stress levels employed in this research were not high and the smaller-sized grains provide a cushion for the large-sized grains, so particles rearrangement could be the prevailing mechanism of volume change and particles crushing possibly has a limited participation.

Figure 6. Variations of axial and radial strains with mean effective stresses a) compaction ratio 85%, b) compaction ratio 87.5%, and c) compaction ratio 90%
5. Shear Behavior and the Critical State

The triaxial compression shearing test was carried out based on the details summarized in Tables 3 and 4 for both the drained and undrained conditions, respectively. Figure 8 indicates the typical stress-strain relation of both the drained and undrained shearing tests. Under the drained conditions, the soil revealed a strain hardening behavior in the low mean effective stress levels 100 and 200 kPa, while a strain softening behavior was present in the higher stress levels 300 and 400 kPa, in which the deviator stress decreased slightly after reaching the peak. Under both the drained and undrained shearing conditions, the soil strength increased significantly as the consolidation stress level increased. Figure 9 indicates the volumetric response during the drained shear, the soil revealed a contractive response, regardless of the mean effective stresses. It is worth to note that the tests were performed on normally consolidated re-compacted specimens, thus the variations of the initial void ratio (i.e., before shearing stage) resulted from different consolidation stresses. The volumetric response under the undrained condition can be indirectly concluded from the stress path shape indicated in Figure 10. For the soil specimens sheared at low consolidated stress level, the soil exhibited firstly contractive response, as the stress path moved to the left direction, and then the soil passed through a phase transformation point at zero dilation, and after that the stress path turned towards the right direction, which is a sign of the tendency to soil dilation. After passing through the phase transformation point, the deviator and mean effective stresses increased with a constant stress ratio until reaching the critical state. The soil specimens sheared at high consolidation stress levels, revealed a slight increase in the mean effective stress at the early stage of shearing. The mean effective stress then decreased significantly after the first phase transformation point. Towards the end of the stress path, the mean effective stress increased again before reaching the critical state and after surpassing the second phase transformation point. Phase transformation points were also observed in the CDG soils reported by Yan and Li [12], Wang and Yan [13] and Lee and Coop [10]. However, no phase transformation revealed for the CDG soil studied by Ng and Chiu [27]. The initial stiffness during the undrained conditions was higher than those in the drained conditions. Nevertheless, the soil exhibited drained strength much more than that in the undrained condition.

The critical state can be reached when the rate of changes in stresses and volume become insignificant. Thus, the critical state of the CDG soil can be estimated by the specimens’ state at 30% axial strain, in which there was no significant change in volume and stress as shown in Figures 8 and 9. The critical states have been represented in both the q-p’ and e-log p’ spaces as depicted in Figures 10 and 11, respectively. The CDG soil showed a unique CSL, irrespective of the drainage conditions and initial states in both the volumetric and stress spaces (i.e., q-p’ and e-log p’).

In the q-p’ space, the CDG soil approached unique CSL at critical stress ratio (M) equal 1.5, corresponding to critical friction angle of 36.8°. In the volumetric space e-log p’, the CDG soil also revealed a unique CSL parallel to the ICL. Therefore, the state parameter proposed by Been and Jefferies [28]; which is defined as the vertical distance between ICL and CSL in the e-log p’ space is constant, regardless of mean effective stress levels. The parallelism of the ICL and CSL was also observed in CDG soil studied by Lee and Coop [10]. However, the CDG soils studied by Yan and Li [12] and Ng, Fung [11] showed a non-parallelism of ICL and CSL, where they tended to converge as the effective mean stress increased, which possibly attributed to soils mineralogy and micro-fabric.
This study presented the results of a set of drained and undrained triaxial shearing and isotropic compression tests on medium-coarse grading CDG soils to investigate their behavior. The experimental results have been interpreted based on the results of the XRD analysis and the scanning electron microscopic-photos. The soil showed low compressible response during the isotropic compression with a unique isotropic compression line. The volumetric change was unrecoverable throughout the unloading path. A 250 kPa effective mean stress was sufficient to erase the induced anisotropy from the moist tamping preparation method, and then the soil showed the isotropic response. After the analysis of the isotropic compression results with the XRD analysis results and the SEM photos, it can be concluded that the soils' mineralogy and micro-fabric are considered paramount factors governing the compression behavior in addition to grading.

The soil exhibited a strain hardening behavior under the undrained conditions, irrespective to the consolidation stress level. Under the drained conditions, the soil revealed a strain hardening at low consolidation stress levels; however, a slight tendency of a strain softening response revealed at high stress levels. The soil exhibited a contractive response under the drained shearing. The undrained response of the soil was complicated, in which the stress path in the low consolidated stress was contractive from the beginning of the shearing and after passing through the phase transformation point, the soil response changed from contractive to dilative, and then both the deviator and mean effective stresses increased with a constant stress ratio until reaching the critical state. In the high consolidation stresses, the soil initially exhibited a dilative response followed by a contractive response, and then a dilative response up to the critical state with two phase transformation points. The soil showed a unique critical state line in both the q-p' and e-log $p'$ spaces, regardless of the drainage conditions and initial states (i.e., void ratio and initial density). The CSL in the e-log $p'$ space was parallel to the ICL. The stress ratio at the critical state was 1.5, corresponding to a critical state friction angle of 36.8°.

### 6. Conclusions

The soil exhibited a strain hardening behavior under the undrained conditions, irrespective to the consolidation stress level. Under the drained conditions, the soil revealed a strain hardening at low consolidation stress levels; however, a slight tendency of a strain softening response revealed at high stress levels. The soil exhibited a contractive response under the drained shearing. The undrained response of the soil was complicated, in which the stress path in the low consolidated stress was contractive from the beginning of the shearing and after passing through the phase transformation point, the soil response changed from contractive to dilative, and then both the deviator and mean effective stresses increased with a constant stress ratio until reaching the critical state. In the high consolidation stresses, the soil initially exhibited a dilative response followed by a contractive response, and then a dilative response up to the critical state with two phase transformation points. The soil showed a unique critical state line in both the q-p' and e-log $p'$ spaces, regardless of the drainage conditions and initial states (i.e., void ratio and initial density). The CSL in the e-log $p'$ space was parallel to the ICL. The stress ratio at the critical state was 1.5, corresponding to a critical state friction angle of 36.8°.
7. Funding

This paper was funded from the National Natural Science Foundation of China (NSFC) (No. 51508216 and 416772267.

8. Conflicts of Interest

The authors declare no conflict of interest.

9. References


